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CRITERIA FOR AIRPORT PAVEMENTS. (U)  
SEP 76 R L HUTCHINSON, H M UERY

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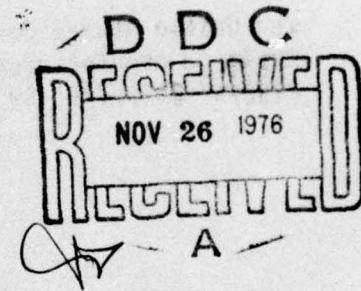
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## CRITERIA FOR AIRPORT PAVEMENTS FINAL SUMMARY REPORT

U. S. Army Engineer Waterways Experiment Station  
Soils and Pavements Laboratory  
P. O. Box 631, Vicksburg, Miss. 39180



SEPTEMBER 1976  
FINAL REPORT

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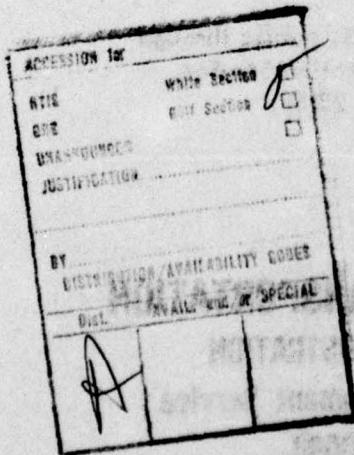
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16. Abstract  An investigation was conducted which resulted in suggested changes to existing Federal Aviation Administration design, construction, and evaluation criteria for civil airport pavements. Through individual studies, design and construction procedures were developed for prestressed concrete, fibrous concrete, and continuously reinforced concrete pavements; porous friction surface courses; and pavements incorporating stabilized layers, insulating layers, and membrane-encapsulated layers. Proposals were made for statistical quality control of paving materials, and a review of soil classification systems applicable to airport pavement design was accomplished. A complete frost and permafrost design procedure for civil airport pavements was formulated. A nondestructive pavement evaluation procedure based on steady state vibratory loadings and measurements of the resulting elastic deflections was developed. An economic analysis was performed to relate pavement upgrading cost to a penalty cost associated with adding gears and wheels to aircraft in order to provide flotation for present-day pavement design criteria. Studies of aircraft dynamic load effects and wheel-path distribution were made. This report summarizes these studies. Individual technical reports are available for all of the above-mentioned studies.			
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## METRIC CONVERSION FACTORS

### Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
<b>LENGTH</b>				
inches	*2.5	centimeters	mm	mm
feet	.30	centimeters	cm	cm
meters	0.3	meters	m	m
kilometers	1.6	kilometers	km	km
<b>AREA</b>				
square inches	6.5	square centimeters	cm <sup>2</sup>	cm <sup>2</sup>
square feet	0.09	square meters	m <sup>2</sup>	m <sup>2</sup>
square yards	0.8	square meters	m <sup>2</sup>	m <sup>2</sup>
square miles	2.5	square kilometers	km <sup>2</sup>	km <sup>2</sup>
acres	0.4	hectares	ha	ha
<b>MASS (weight)</b>				
ounces	28	grams	g	g
pounds	0.45	kilograms	kg	kg
short tons	0.9	tonnes	t	t
(2000 lb)				
<b>VOLUME</b>				
tablespoons	5	milliliters	ml	ml
teaspoons	15	milliliters	ml	ml
fluid ounces	.30	liters	l	l
cups	0.24	liters	l	l
pints	0.47	liters	l	l
quarts	0.95	liters	l	l
gallons	3.8	cubic meters	m <sup>3</sup>	m <sup>3</sup>
cubic feet	0.03	cubic meters	m <sup>3</sup>	m <sup>3</sup>
cubic yards	0.75	cubic meters	m <sup>3</sup>	m <sup>3</sup>
<b>TEMPERATURE (exact)</b>				
Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C	°C

### Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
<b>LENGTH</b>				
inches	0.04	inches	in	in
centimeters	0.4	inches	in	in
meters	3.3	feet	ft	ft
meters	1.1	yards	yd	yd
kilometers	0.6	miles	mi	mi
<b>AREA</b>				
square inches	0.16	square inches	in <sup>2</sup>	in <sup>2</sup>
square centimeters	1.2	square yards	yd <sup>2</sup>	yd <sup>2</sup>
square meters	0.4	square miles	mi <sup>2</sup>	mi <sup>2</sup>
square kilometers	2.5	hectares (10,000 m <sup>2</sup> )	ha	ha
<b>MASS (weight)</b>				
ounces	0.035	ounces	oz	oz
pounds	2.2	pounds	lb	lb
kilograms	1.1	short tons	t	t
tonnes (1000 kg)				
<b>VOLUME</b>				
fluid ounces	0.03	fluid ounces	fl oz	fl oz
pints	2.1	pints	pt	pt
quarts	1.06	quarts	qt	qt
liters	0.26	gallons	gal	gal
cubic meters	35	cubic feet	cu ft	cu ft
cubic meters	1.3	cubic yards	cu yd	cu yd
<b>TEMPERATURE (exact)</b>				
Celsius temperature	5/9 (then add 32)	Fahrenheit temperature	°F	°F

\*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Spec. Publ. 286, Units of Weight and Measures, Price \$2.25, SD Catalog No. C13-10286.

## PREFACE

This investigation was conducted by the Soils and Pavements Laboratory, U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., for the U. S. Department of Transportation, Federal Aviation Administration, under Inter-Agency Agreement No. DOT FA71WAI-218. This report covers work done from May 1971-December 1975.

The program was conducted under the general supervision of Mr. James P. Sale, Chief of the Soils and Pavements Laboratory, and Mr. Ronald L. Hutchinson, Pavement Program Manager. The following WES consultants participated in the conduct of the program: Prof. R. E. Fadum, North Carolina State University; Prof. M. E. Harr, Purdue University; Prof. W. R. Hudson, The University of Texas; Prof. W. H. Goetz, Purdue University; Prof. C. L. Monismith, University of California; and Mr. W. J. Turnbull, Vicksburg, Miss.

The individual chapters comprising this report were prepared by Soils and Pavements Laboratory (S&PL) project engineers and were reviewed and compiled by Messrs. Hutchinson and Harry H. Ulery, Jr., Chief of the Pavement Design Division.

During the conduct of the program, Directors of WES were BG E. D. Peixotto, CE, and COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

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## CHAPTER 1: INTRODUCTION

### 1.1 BACKGROUND

The jet age in air transport was inaugurated in 1958 when the Boeing 707 entered commercial service. Subsequent rapid developments in the field in the 1960's, particularly with regard to increased traffic and greater aircraft gross weights, resulted in more stringent airport operational requirements and indicated the need to update existing technical design and construction criteria. During this period, few changes were made in civil airport pavement design criteria although the jet aircraft with their heavier wheel loads and higher traffic intensities dictated the need for stronger and smoother airport pavements. Pavement deterioration became more evident and began to present serious problems.

The Federal Aviation Administration (FAA) has primary responsibility for insuring that safe, economical, and reliable design and construction standards for civil airport pavements are established and maintained. The studies described in this report were undertaken as part of this continuous program.

### 1.2 PURPOSE

The purpose of this report is to summarize the results of various studies conducted as part of the FAA's multiphase program for updating design and construction procedures and criteria for civil airport pavements.

### 1.3 SCOPE

This part of the multiphase program was accomplished by conducting 14 individual studies under three general areas: design criteria, materials and construction, and test and evaluation. Technical reports generated by these studies are listed in Table 1-1.

### 1.4 HISTORY AND CHRONOLOGY

On 26 May 1971, Inter-Agency Agreement No. DOT FA71WAI-218, "Development of Airport Pavement Criteria," between the U. S. Department

Table 1-1

Technical Reports Generated Under Inter-Agency Agreement No. DOT FA71WAI-218

Number	Report Number	Title
1	FAA-RD-72-106	Strengthening of Keyed Longitudinal Construction Joints in Rigid Pavements
2	FAA-RD-72-119	Construction of Fibrous Reinforced Concrete Overlay Test Slabs, Tampa International Airport, Florida
3	FAA-RD-73-33-I	Continuously Reinforced Concrete Airfield Pavement; Tests on Existing Pavements and Synthesis of Design Methods
4	FAA-RD-73-33-II	Continuously Reinforced Concrete Airfield Pavement; Design Manual for Continuously Reinforced Concrete Overlay Pavements
5	FAA-RD-73-33-III	Continuously Reinforced Concrete Airfield Pavement; Design Manual for Continuously Reinforced Concrete Pavements
6	FAA-RD-73-33-IV	Continuously Reinforced Concrete Airfield Pavement; Guide Specification
7	FAA-RD-73-37	Design and Construction of Continuously Reinforced Concrete Airport Pavements
8	FAA-RD-73-169	Review of Soil Classification Systems Applicable to Airport Pavement Design
9	FAA-RD-73-197	Porous Friction Surface Course
10	FAA-RD-73-198-I	Comparative Performance of Structural Layers in Pavement Systems; Design, Construction, and Behavior Under Traffic of Pavement Test Sections
11	FAA-RD-73-198-II	Comparative Performance of Structural Layers in Pavement Systems; Analysis of Test Section Data and Presentation of Design and Construction Procedures
12	FAA-RD-73-198-III	Comparative Performance of Structural Layers in Pavement Systems; Design and Construction of MESL
13	FAA-RD-73-198-IV	Comparative Performance of Structural Layers in Pavement Systems; Analysis of Insulating Layers in Pavement Test Sections
14	FAA-RD-73-199	Statistical Quality Control Procedures for Airport Pavement Materials
15	FAA-RD-73-205-I	Nondestructive Vibratory Testing of Airport Pavements; Experimental Test Results and Development of Evaluation Methodology and Procedure
16	FAA-RD-73-205-II	Nondestructive Vibratory Testing of Airport Pavements; Theoretical Study of the Dynamic Stiffness and Its Application to the Vibratory Nondestructive Method of Testing Pavements
17	FAA-RD-73-206	Aircraft-Pavement Compatibility Study
18	FAA-RD-74-30	Design of Civil Airfield Pavements for Seasonal Frost and Permafrost Conditions
19	FAA-RD-74-31	Steel Fibrous Concrete for Airport Pavement Applications
20	FAA-RD-74-34-I	Prestressed Concrete Pavements; Dulles Test Road Instrumentation and Load Tests
21	FAA-RD-74-34-II	Prestressed Concrete Pavements; Design and Construction Procedures for Civil Airports
22	FAA-RD-74-35	Criteria for Airport Pavements; Final Summary Report
23	FAA-RD-74-36	Field Survey and Analysis of Aircraft Distribution on Airport Pavements
24	FAA-RD-74-38	Field Performance of Porous Friction Surface Course
25	FAA-RD-74-39-I	Pavement Response to Aircraft Dynamic Loads; Instrumentation Systems and Testing Program
26	FAA-RD-74-39-II	Pavement Response to Aircraft Dynamic Loads; Presentation and Analysis of Data
27	FAA-RD-74-39-III	Pavement Response to Aircraft Dynamic Loads; Compendium

of Transportation, FAA, and the U. S. Army Engineer Waterways Experiment Station (WES), was signed for the purpose of initiating a program for development of airport pavement criteria in accordance with Engineering Requirement No. FAA-ER-D-160-006, dated 31 March 1971.

Subsequent to the signing of the inter-agency agreement, the proposed pavement research program was reviewed at a WES consultants' conference which was held at WES on 8-10 June 1971. Afterwards, a draft report was formulated at a special consultants' meeting held at WES on 6-7 July 1971. The consultants' report was completed and distributed on 25 August 1971.

The basic inter-agency agreement was amended on 22 May 1972 with the incorporation of Amendment No. 1 to FAA-ER-D-160-006, dated 10 March 1972. Since this time, the studies comprising the engineering requirement have been as listed in Table 1-2.

During the conduct of this part of the program, numerous meetings have been held for the purpose of reviewing progress on the various studies. The most significant of these meetings were held at WES on the following dates: 8-10 June 1971 (with sponsors, industry representatives, and WES consultants); 26-30 June 1972 (with sponsors, industry representatives, and WES consultants); 8-9 February 1973 (with sponsors and industry representatives); 13-19 July 1973 (with sponsors, industry representatives, and WES consultants); 16-20 July 1974 (with sponsors); 1-2 August 1974 (with sponsors, sponsors' consultants, and industry representatives); and 25-26 June 1975 (with sponsors, sponsors' consultants, and industry representatives).

Table 1-2  
Studies Comprising Engineering Requirement No. FAA-ER-D-160-006

Area of Study	Title of Study	Contributing Personnel	Date Work Started	Conducted By
Design Criteria	Prestressed Concrete Pavements	Mr. E. C. Odom Mr. D. M. Ladd Mr. R. H. Ledbetter Dr. F. Parker, Jr. Mr. P. E. Carlton, OCE	Aug 71	WES in conjunction with the Federal Highway Administration
	Steel Fibrous Concrete Pavements	Dr. F. Parker, Jr.	Sep 71	WES*
	Continuously Reinforced Concrete Pavements	Dr. F. Parker, Jr. Mr. G. G. Harvey	Sep 71	WES and the Air Force Weapons Laboratory under contract
	Frost and Permafrost	Dr. R. L. Berg, CRREL Dr. G. M. Hammitt II	May 73	Cold Regions Research and Engineering Laboratory under contract to WES
	Aircraft Distribution on Pavements	Dr. W. J. Horn Mr. D. N. Brown Mr. V. HoSang, HNTB	May 72	Howard, Needles, Tammen, and Bergendoff under contract to WES
	New Criteria for Pavement Design and Construction	Mr. D. M. Ladd Mr. D. L. Cooksey Mr. G. G. Harvey Prof. E. J. Yoder, Purdue	Jan 72	WES and partly under contract to Prof. E. J. Yoder
Materials and Construction	Porous Friction Surface Courses	Mr. T. D. White	Dec 71	WES
	Stabilized Layers, Insulating Layers, and Membrane-Encapsulated Soil Layers	Mr. C. D. Burns Mr. W. N. Brabston Dr. G. M. Hammitt II Dr. W. R. Barker Mr. C. L. Rone Dr. F. Parker, Jr. Mr. D. M. Ladd Mr. S. L. Webster	Aug 72	WES*
	Statistical Quality Control of Paving Materials and Construction	Mr. E. R. Brown Mr. T. D. White	Oct 72	WES
Test and Evaluation	Pavement Response to Aircraft Dynamic Loads	Mr. R. H. Ledbetter Dr. W. J. Horn	May 71	WES in conjunction with the FAA's National Aviation Facilities Experimental Center
	Load-Transfer Mechanisms for Joints in Rigid Pavement	Mr. R. W. Grau Mr. C. D. Burns	Jul 71	WES**
	Aircraft-Pavement Compatibility	MAJ F. H. Griffis, Jr. Mr. T. D. White Mr. M. A. Gamon, Lockheed Mr. P. Durup, Lockheed	Nov 71	WES and the Lockheed-California Company under contract
	Nondestructive Testing	Mr. J. W. Hall, Jr. Mr. R. A. Weiss Mr. J. L. Green Dr. F. Parker, Jr. Mr. D. M. Ladd Dr. G. M. Hammitt II Mr. H. R. Austin Dr. W. R. Barker	Jun 72	WES

\* Jointly sponsored by the Office, Chief of Engineers, U. S. Army (OCE), and the FAA.  
 \*\* Jointly sponsored by the Air Force Weapons Laboratory and the FAA.

## CHAPTER 2: PRESTRESSED CONCRETE PAVEMENTS

### 2.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-74-34, "Prestressed Concrete Pavements." The report is contained in two volumes. Volume I is entitled "Dulles Test Road Instrumentation and Load Tests," October 1974, and Volume II is entitled "Design and Construction Procedures for Civil Airports," November 1974.

### 2.2 BACKGROUND

Prestressing to strengthen concrete has been used widely and successfully for bridges, buildings, storage tanks, and pressure pipes in the past 25 years; however, only a modest interest and limited investment of research funds have been directed at prestressed concrete as a pavement, particularly for airports. As a result, the current state of the art in the design and construction of such pavements is not highly developed. Since the first prestressed concrete pavement on record was constructed on a bridge approach at Luzancy, France, in 1946, only about 100 prestressed pavement test sections and test slabs are known to have been constructed. These sections have been about evenly divided between airports and highways. The most recently constructed sections of prestressed pavement were on an access road to Dulles International Airport and in Pennsylvania. These were designed and constructed by the Federal Highway Administration and the Pennsylvania Department of Transportation, respectively.

Gross weights of current and proposed commercial aircraft have reached such proportions and flight operations have reached such intensities that as much as 16 in. or more of plain concrete may be required to provide an adequate pavement. In view of recent increased concern over a more effective use of the Nation's resources, there is a basis for renewed interest in the search for resource saving methods of constructing pavements. The desire to evaluate thoroughly the possible expanded role of prestressed pavements stems primarily from three basic advantages such pavements offer over conventional rigid pavements.

First, it has been demonstrated, both analytically and in testing, that prestressed pavements permit a substantial reduction in pavement thickness (50 percent or more), with corresponding savings in construction materials and possibly costs. Second, prestressed pavements can be designed with fewer joints, a characteristic which results in quieter and smoother rides and eliminates the need for costly sealing and re-sealing programs. Third, the lesser number of joints and lower probability of crack formation (both load and nonload associated) can be projected into the likelihood of extended pavement life and reduced maintenance requirements.

There are, however, potential disadvantages associated with prestressed pavements which must also be considered. First, there is an increase in the complexity of construction which leads to higher costs offsetting the savings of materials. There are strong indications that improved construction techniques which will result from increased usage of prestressed pavements will help minimize these costs. Second, the joints that are required in a prestressed pavement are expensive to construct and, due to the large horizontal movements, are difficult to maintain. However, through the use of improved materials and construction techniques, more durable joint systems will probably be developed.

Data from only a relatively few full-scale test pavements, laboratory tests on small-scale models, and observations of the performance of a limited number of operational airport pavements are available as a basis for extending and refining the design and construction procedures. A review of the approaches to design employed in the construction of the various prestressed pavement test sections revealed that few attempts have been made to develop these designs by analytical techniques. Generally, pavement thicknesses and amounts of prestress have been selected on an arbitrary basis. Most highway pavement test sections have been 6 in. thick with only longitudinal prestressing, while airport pavement thicknesses have reached 9 in. with both longitudinal and transverse prestressing. Frequently, such empirical designs

have been subjected to static load tests following completion of the construction in order to evaluate the load-carrying capability of the pavement. Also, frequent attempts have been noted at making quantitative assessments for such design parameters as: (a) the effects of hygrothermal stresses, (b) subgrade restraint, and (c) prestress losses associated with the stressing tendons and anchorage systems. Other than in studies conducted by the Corps of Engineers, little evidence was found to indicate that the effect of frequency of load applications has been considered in design.

#### 2.3 PURPOSE

The purpose of this study was to develop suitable procedures based on available data for the design and construction of prestressed pavements at airports serving the civil aviation community.

#### 2.4 SCOPE

The study included: (a) review of technical literature describing the construction, testing, and performance of prestressed concrete pavements, (b) selection of the design criteria that have been best validated by experimentation, (c) formulation of a design procedure based on the selected criteria, and (d) description of recommended construction procedures. In addition, load-deflection measurements were made on the Dulles International Airport prestressed highway test road in an effort to further develop or verify the design criteria.

#### 2.5 SUMMARY OF WORK ACCOMPLISHED

In the review of previous research, studies pertaining to prestressed highway as well as airport pavements were included. However, because of the differences in design requirements for highway and airport pavements, primary consideration was given to research pertaining to airport pavements. In the mid 1950's, the Corps of Engineers at its Ohio River Division Laboratories (ORDL) conducted a research program consisting of theoretical studies, model studies, and full-scale test sections that resulted in the design and construction of a

prestressed concrete pavement for a heavy-load taxiway at Biggs Air Force Base, Texas, in 1959. This pavement has performed well except for some problems at the joints, which are spaced on 500-ft centers. The design procedure developed at ORDL was selected as the basis for the design procedure developed during this study.

One component of the ORDL design procedure involved predicting load-stress relationships based on small-scale tests using the gear configurations of specific military aircraft. However, present-day commercial aircraft have gear configurations that are different from those used in developing the design procedure. Thus, it was necessary to develop load-stress relationships for present-day commercial aircraft. This was accomplished using a computer program based on a discrete element procedure for plates and slabs. The data obtained from previous small-scale models were used to establish the necessary input parameters for the computer model. With this modification, the design procedure was adapted for the standard dual and dual-tandem gear aircraft now operating at civil airports and also for the newer wide-body jet aircraft. The final FAA design procedure permits interrelating magnitude of loading, load repetitions, flexural strength, subgrade conditions, pavement thickness, slab dimensions, and amount of prestress. Consideration is also given to the effects of elastic shortening, creep, and shrinkage of concrete, relaxation in steel tendons, anchorage systems, tendon friction, subgrade restraint, and temperature changes.

In an effort to further validate and refine the design criteria, full-scale static and moving load tests were conducted on the prestressed concrete test road near Dulles International Airport, Va. This prestressed highway pavement was constructed by the Federal Highway Administration as part of the airport road network serving the 1972 International Exposition (Transpo 72). Strain gages and pressure cells were installed within the pavement structure during construction in two separate prestressed concrete slabs. Tests were conducted on the two instrumented slabs using a truck to represent highway loads and a load cart equipped with one dual-tandem component of a B-747 aircraft to represent aircraft loadings. These tests consisted of measurements of

stress and strain in the prestressed concrete pavement structure under various loading conditions.

The load tests conducted on the Dulles test road showed good correlation between measured subgrade behavior directly beneath the loads and that determined using linear elastic layer theory. For the subgrade conditions, pavement slab conditions, and load conditions at Dulles, this correlation indicates that the subgrade can be modeled by elastic layer theory. The load tests were deliberately held all within the initial prefailure behavior of the pavements so that no cracks or failures would occur. The results of these tests demonstrated that the initial maximum elastic deflections and stresses of the pavement slab may be closely approximated by linear elastic layer theory. The results of these tests also indicated that the maximum subgrade deflections or deformations determined using the layer theory may be used for calculating the slab bending moments and stresses by the various slab behavior models. The layer theory model results can also be used with Westergaard's analysis and his correction factors based on measured deflections. Layer theory deflections could be specifically incorporated in Westergaard's subgrade reaction corrections. The above discussion applies only to the Dulles test pavements; future work should further investigate modeling the subgrade by linear elastic layer theory.

Construction techniques and alternatives were based on examination of prototype test pavements and operational prestressed facilities constructed in this country and abroad. Special emphasis was given to developing an expansion joint which could withstand the relatively large daily and seasonal movements of the prestressed concrete slab ends that occur due to the increased length of prestressed slabs as compared with conventional concrete slabs. After investigations of past construction projects and discussions with manufacturers, several alternative types of joints and joint materials were selected for inclusion in the construction procedure. Other construction recommendations included assessments of the relative merits of prestressing with and without

tendons, pretensioning versus posttensioning, types of stressing tendons and conduits, friction reducing layers, and amounts of prestressing.

#### 2.6 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Prestressed concrete pavements can be designed for airport pavements with a reasonable degree of accuracy with respect to the load-carrying capability and the number of load repetitions that can be sustained.

Construction of airport pavements with prestressed concrete rather than conventional concrete will result in a savings of concrete due to the smaller thicknesses required. In addition, the long prestressed slabs will result in fewer joints to be maintained and will provide a smoother operating surface.

The design criteria and construction procedures are considered to be conservative in some areas. This is due to the uncertain state of the art in these areas. All recommendations are subject to refinement pending further study.

The following experimental work is recommended to refine the design procedure and to improve its reliability:

- a. The relationship between the ratio of the loaded area to the pavement radius of relative stiffness and the percent single-wheel failure load (the ratio, expressed as a percentage, of the load on a single tire to the load on a multiple-wheel gear which would cause cracking in the prestressed pavement) plays an important role in the selection of prestress level and pavement thickness and should be verified before widespread use of the criteria is made. This verification should be by means of model tests using procedures similar to those used in the earlier tests by ORDL.
- b. Sections of instrumented prestressed pavement should be constructed for a range of foundation types and tested to gain verification of the design criteria presented and to modify the criteria as necessary.

## CHAPTER 3: STEEL FIBROUS CONCRETE PAVEMENTS

### 3.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-72-119, "Construction of Fibrous Concrete Overlay Test Slabs, Tampa International Airport, Florida," October 1972, and Report No. FAA-RD-74-31, "Steel Fibrous Concrete for Airport Pavement Applications," November 1974.

### 3.2 BACKGROUND

Fibrous concrete is a composite material consisting of a concrete matrix containing a random dispersion of small fibers. Numerous types of fiber materials have been investigated, including steel, fiber glass, nylon, asbestos, polypropylene, and polyethylene. The introduction of fibers into the concrete matrix imparts to the concrete certain characteristics such as increased tensile strength, increased toughness, increased impact and dynamic strength, increased resistance to spalling, resistance to propagation of cracks, and the ability to sustain load and keep cracks tightly closed after cracking.

For pavement applications, steel fibers have been used almost exclusively. As considered in this chapter, the term "fibrous concrete" will be synonymous with concrete containing steel fibers; general usage of the term "fibrous concrete" implies the use of any of a number of fiber material types.

For comparable design loadings, the required thickness of fibrous concrete pavement is less than that of plain or conventionally reinforced pavement. In situations where a thin pavement is necessary because of such factors as vertical grade or drainage considerations or in areas where there is a shortage of quality aggregate or other material, it may be advantageous and become economically feasible to use fibrous concrete.

### 3.3 PURPOSE

The purpose of this study was to develop a design procedure for fibrous concrete airport pavements based on known properties of fibrous

concrete mixtures. In addition to the development of a design procedure, guidance for mix proportioning and construction of fibrous concrete pavements was to be provided.

### 3.4 SCOPE

The study involved (a) construction and testing of four full-scale pavement sections under controlled, accelerated traffic conditions; (b) planning and construction of two field placements; and (c) monitoring efforts of other agencies. The design criteria developed and the recommended construction practices reflect the findings from these tests and observations.

Since only four pavement sections were tested, it was necessary to extrapolate the performance from three slab-on-grade sections to all other foundation conditions and from a partial bond overlay to all other overlay conditions. It was also necessary to assume that the long-term field performance would be comparable to the performance under accelerated test conditions. Recommended construction practices were developed from experience gained during the construction of relatively small quantities of pavement.

### 3.5 SUMMARY OF WORK ACCOMPLISHED

The controlled accelerated pavement sections included the following: (a) a 6-in. slab over a 4-in. sand filter on a clay subgrade; (b) a 7-in. slab over a 20-in. membrane-encapsulated layer of lean clay on a clay subgrade; (c) a 4-in. slab over a 17-in. cement-treated clay gravel base on a clay subgrade; and (d) a 4-in. partially bonded overlay of a failed 10-in. plain concrete pavement. Applied traffic consisted of simulated C-5A and B-747 loadings. Figures 3-1 to 3-3 illustrate the deterioration of the 4-in.-thick slabs as simulated B-747 traffic was applied. From results of these tests, the design criteria were developed.

Field installations investigated included the following: (a) 4- and 6-in. overlay sections constructed on a taxiway at Tampa International Airport and (b) a 4-in. roadway constructed at WES. The Tampa

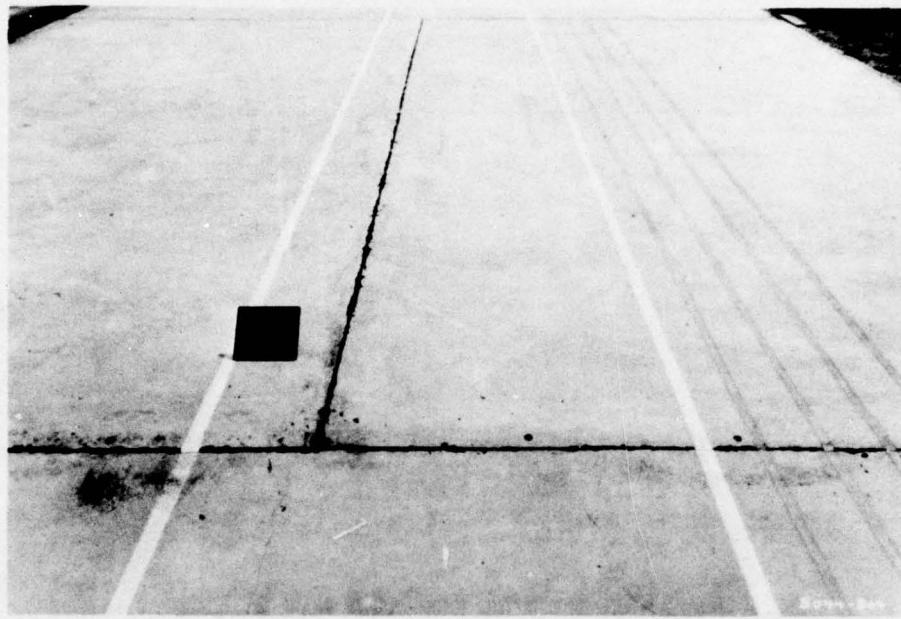


Figure 3-1. Condition of 4-in. fibrous concrete slab  
on 17-in. cement-treated base before traffic

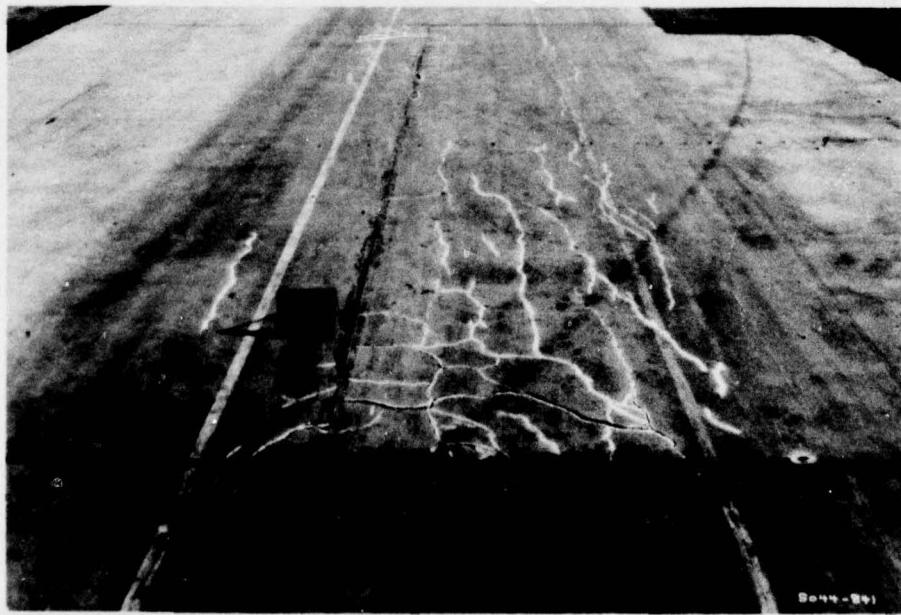


Figure 3-2. Condition of 4-in. fibrous concrete slab  
on 17-in. cement-treated base after 1000 coverages of  
simulated B-747 traffic

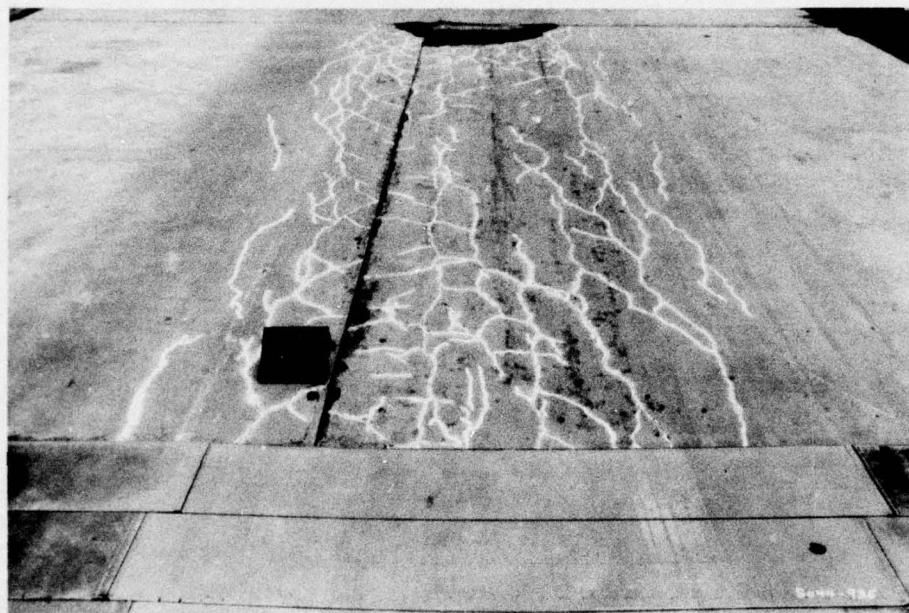


Figure 3-3. Condition of 4-in. fibrous concrete slab  
on 17-in. cement-treated base after 3000 coverages of  
simulated B-747 traffic

overlays were constructed with conventional paving equipment to determine the feasibility of constructing fibrous concrete pavements with this type of equipment. Figures 3-4 and 3-5 illustrate the construction techniques used. Figure 3-6 illustrates the crack pattern in the base pavement and the crack pattern that developed in the overlay after 6 and 28 months in service. The roadway was constructed at WES to study joint requirements for fibrous concrete pavement. A 1000-ft-long section was constructed without provisions for joints. Seven cracks formed in the pavement resulting in slabs 146, 63, 87, 153, 161, 70, 90, and 240 ft in length. The average slab length was 126 ft, indicating that joints on about 100-ft spacings should be used.

An effort has been made to stay abreast of work being conducted by other agencies on material characteristics of fibrous concrete and on its use as a paving material. Recently, work has been done primarily on mix design and on fatigue and durability characteristics. Other agencies have concentrated their efforts toward the use of fibrous concrete for overlays, particularly as overlays of highway pavements. In the past several years, there have been a number of trial placements of thin overlays of highway pavements. Through observations of the efforts of other agencies, additional information concerning the properties and uses of fibrous concrete has been obtained.

### 3.6 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Significant conclusions from this study are summarized as follows:

- a. Fibrous concrete pavements and overlays will perform better than plain concrete pavements of comparable thickness and strength. This means that, for comparable design conditions, the required thickness of fibrous concrete would be less than the required thickness of plain concrete.
- b. The reduced thickness requirements for fibrous concrete will result in increased vertical deflection of the pavement and as a result increased induced stresses in the underlying material. Provisions are made in the design procedure to limit the deflection of the pavement to minimize this effect.
- c. Fibrous concrete can be produced and placed with conventional batching, mixing, and paving equipment and techniques. Bulk handling of fibers and a mechanical system for introducing



Figure 3-4. Introduction of fibers during batching of materials for fibrous concrete

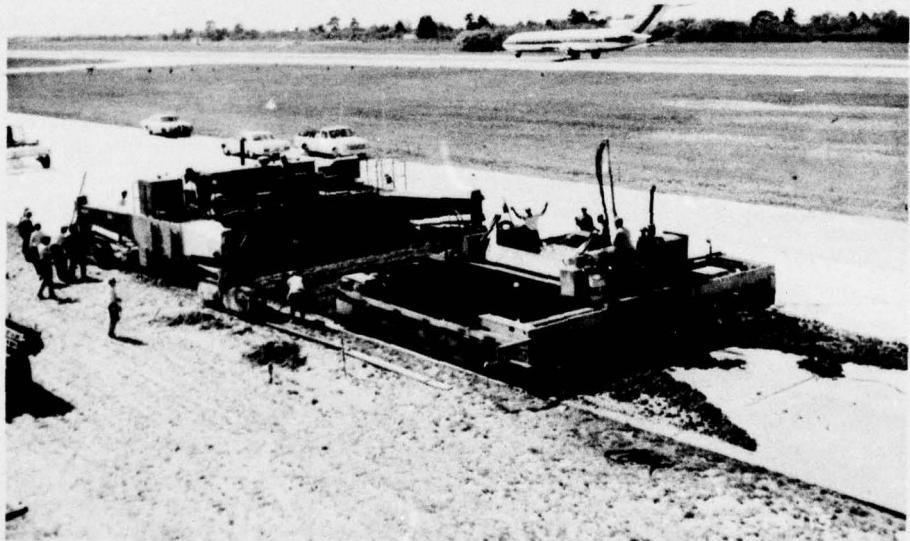
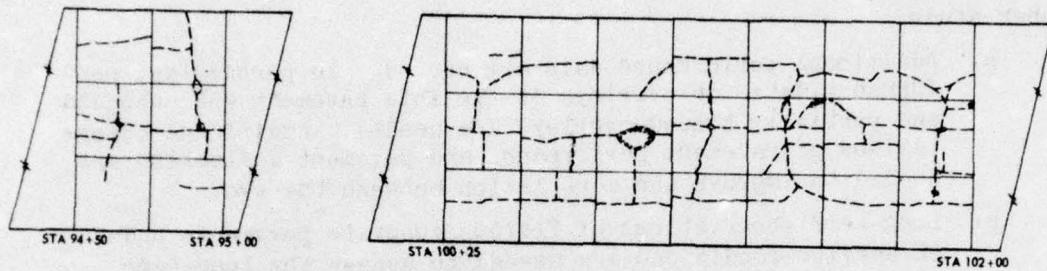
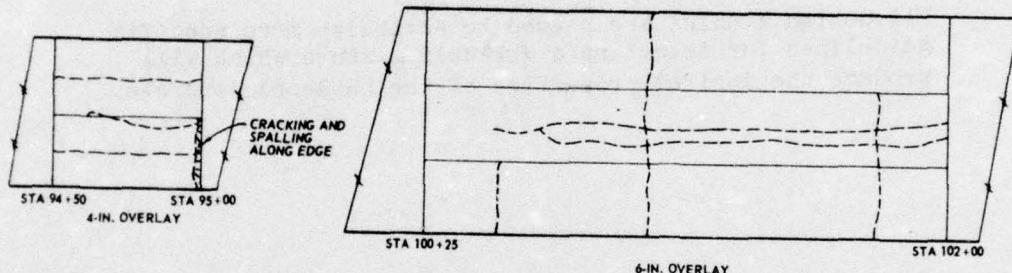


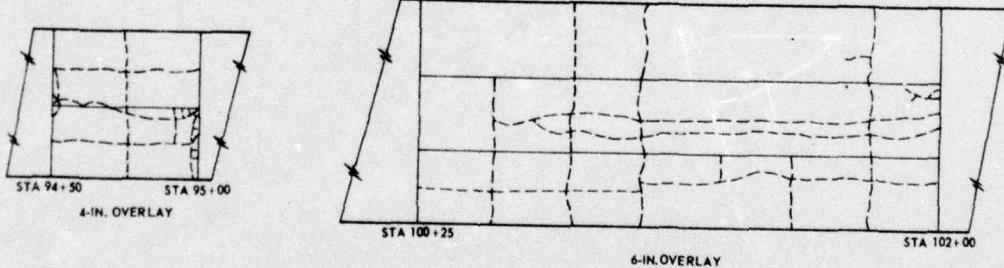
Figure 3-5. Placing fibrous concrete with slip-form paving equipment



a. JOINT AND CRACK PATTERN IN BASE PAVEMENT



b. CRACK PATTERN IN OVERLAYS-6 MONTHS



c. CRACK PATTERN IN OVERLAYS-28 MONTHS

Figure 3-6. Joint and crack pattern in base pavement and overlays for the Tampa overlays

the fibers during batching operations will be required to produce fibrous concrete in sufficient quantities for large airport paving jobs.

In order to improve the reliability of the proposed design criteria and construction techniques, the following areas should receive further study:

- a. Additional performance data are needed. In particular, performance data for overlays of flexible pavement and unbonded and partially bonded overlays are needed. Additional observations of pavement performance and pavement deflection are needed to improve the correlation between the two.
- b. Long-term observations of fibrous concrete pavements under in-service conditions are needed to assess the long-term effects of environmental factors on the performance of fibrous concrete pavements.
- c. Mix design studies are needed to establish more specific guidelines for selecting a workable mixture which will produce the desired properties of the hardened concrete.

## CHAPTER 4: CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

### 4.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-73-33, "Continuously Reinforced Concrete Airfield Pavement," May 1974, and Report No. FAA-RD-73-37 (in preparation), "Design and Construction of Continuously Reinforced Concrete Airport Pavements." Report No. FAA-RD-73-33 is contained in four volumes. These are: Volume I, "Tests on Existing Pavements and Synthesis of Design Methods," Volume II, "Design Manual for Continuously Reinforced Concrete Overlay Pavements," Volume III, "Design Manual for Continuously Reinforced Concrete Pavements," and Volume IV, "Guide Specifications."

### 4.2 BACKGROUND

Continuously reinforced concrete (CRC) pavement is defined as a portland cement concrete pavement with longitudinal reinforcing steel continuous for its length, and in which no transverse joints other than construction joints are installed. Transverse cracks develop in CRC but are held tightly closed by the steel reinforcement. The resulting riding surface generally is smoother and the problems associated with sealing and maintenance of transverse joints are eliminated. For these reasons, it may be advantageous and economical to use CRC for airport pavements.

CRC pavements and overlays have been used on highways for a number of years. They have also been used on airports at a few locations. The most extensive use of CRC has been at O'Hare International Airport and Midway Airport in Chicago, Ill., and at U. S. Air Force Plant 42, Palmdale, Calif. For highway pavements, the various State highway departments have developed design and construction procedures tailored for their particular conditions. These procedures have been used extensively in the development of the design and construction procedures for airport pavements.

#### 4.3 PURPOSE

The purpose of this study was to develop design procedures for airport CRC pavements and overlays. The design procedures include methods for selecting slab thickness, methods for designing the reinforcing steel, and methods for controlling slab end movements. In addition, guidance for the construction of CRC pavements and overlays was to be provided.

#### 4.4 SCOPE

This study was accomplished both in-house by WES personnel and through a joint Air Force-WES contract with Austin Research Engineers, Inc. (ARE), Austin, Tex. The City of Chicago, Ill., through the Bureau of Engineering, provided support for a field study at O'Hare International Airport. The study involved (a) evaluations of existing airport CRC pavements and overlays; (b) a synthesis of data and methods for design of CRC pavements and overlays; (c) formulation of design procedures and construction specifications for CRC airport pavements and overlays based on the evaluations of the existing pavements and existing design methodology; (d) collection of response and performance data for a CRC pavement subjected to actual aircraft loadings and environmental conditions; and (e) from the results of the entire study, development of an implementable design procedure for CRC pavements and overlays which is compatible with FAA procedures for other types of pavement.

#### 4.5 SUMMARY OF WORK ACCOMPLISHED

The study of existing airport CRC pavements and overlays involved data collection at the following locations:

- a. U. S. Air Force Plant 42, Palmdale, Calif.
- b. O'Hare International Airport, Chicago, Ill.
- c. Midway Airport, Chicago, Ill.
- d. Byrd International Airport, Richmond, Va.

The data collected from all four locations consisted of dynamic load-deflection response measurements, characterization of the material

composing the pavements, and collection of pavement condition data. An example of the load-deflection data collected is illustrated in Figure 4-1, which shows the deflection obtained along runway 9R-27L at O'Hare using a Dynaflect vibrator and the WES 16-kip vibrator. The material characterization portion of the study involved collection of samples of the pavement material (disturbed and undisturbed), and testing in the laboratory to determine strength and load-deflection properties of the material. Data from the condition surveys included crack spacings, crack widths, percentage and condition of spalled cracks, longitudinal cracking, and joint condition.

The data from the study of existing airport CRC pavements and overlays were combined with data and design methods from other sources, and tentative design procedures formulated. In addition, specifications for construction of these pavements were proposed.

The proposed design procedures consider stochastic variations in material properties and load location. The variability of material properties is translated into a reliability which can be attached to the resulting pavement. Variability in the loading is directly considered by assuming that the loads will be normally distributed transversely across the pavement. Thus, the number of loads applied to any transverse pavement segment can be determined. The proposed design procedure considers the total mixture of aircraft operating on the facility by considering the pavement damage caused by each different type aircraft, or conversely by considering the thickness required by each different type aircraft. The effects of each aircraft and the variability of the location of each aircraft are combined and added to produce a total thickness requirement which varies transversely across the pavement.

Separate procedures are provided for the design of overlays and for slabs on grade. The elastic layered model is used as the response model for the development of the overlay design procedure, while a slab on a dense liquid foundation is used as the basic response model for the pavement design procedure. The use of different response models requires that different procedures be used for defining input for

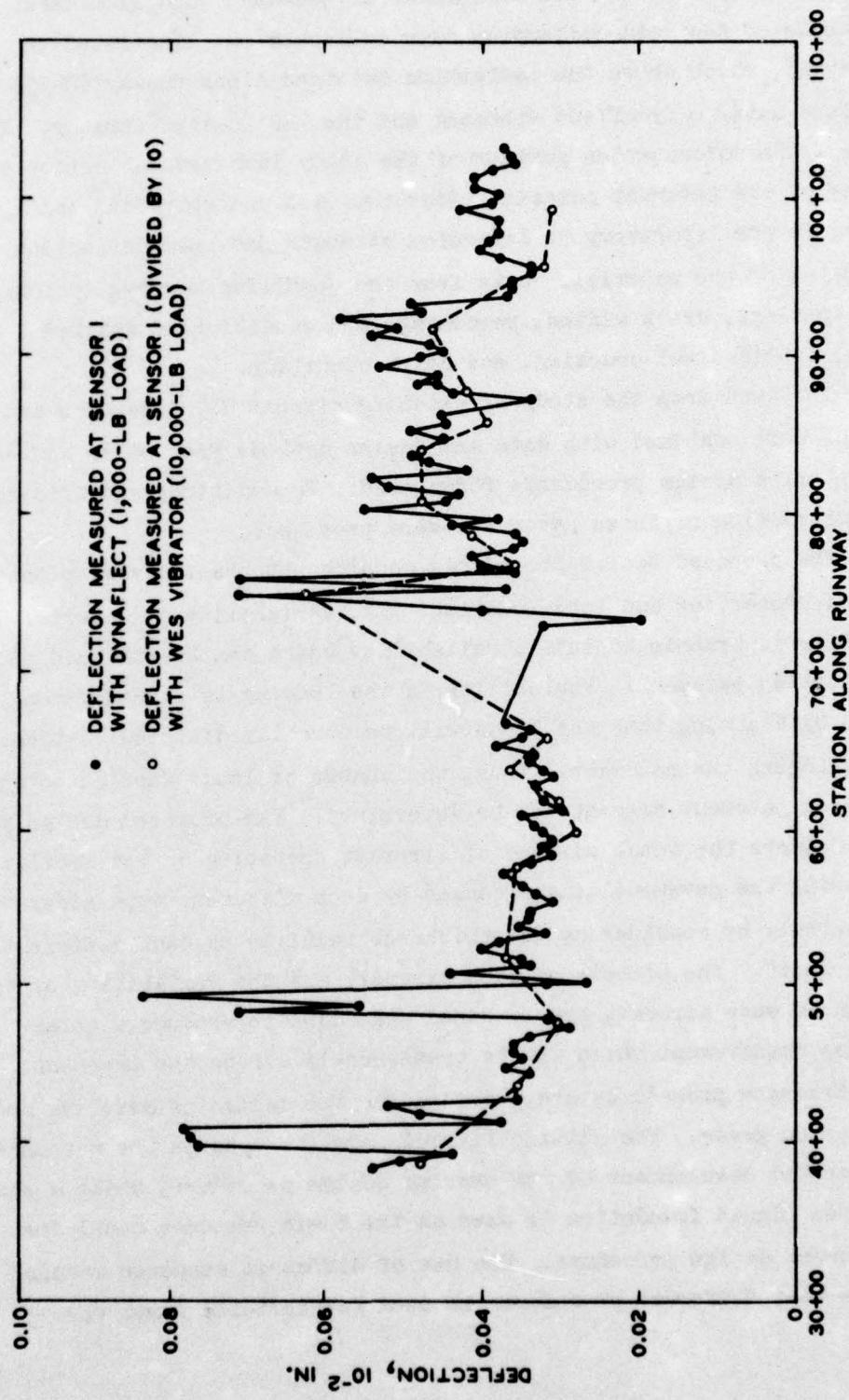


Figure 4-1. Measured dynamic deflections along runway 9R-27L at O'Hare International Airport

characterizing the response of the underlying material. For overlay design, the material in each layer is characterized by two constants (the modulus of elasticity and Poisson's ratio). For pavement design, the response of all layers below the slab is defined by one constant referred to as the composite foundation modulus or "k-value." The procedures for characterizing the load-deflection response and strength of the portland cement concrete are the same, as are the remainder of the procedures for determining required overlay slab thickness.

The design of the steel reinforcement is accomplished by procedures relatively well established and widely used. In addition, recommendations concerning jointing and terminal treatment system designs are provided.

Gages were installed on runway 4R-22L at O'Hare International Airport for measuring the load-deflection responses of the pavement. Bison coils were installed for measuring the strain in the various layers, and LVDT's were installed for measuring the total deflection of the pavement when loaded. Gages of both types were installed at four locations along the runway. In addition to the gages for measuring the load-deflection responses, thermistors were installed to measure slab temperature and reference plugs were installed to be used with a Whitmore strain gage for measuring the opening and closing of transverse cracks with temperature changes.

Initial load-deflection response measurements were made in June 1973. The pavement deflection was measured with LVDT gages with the loads (B-727-100 aircraft and a 125-kip aircraft tug) located at various positions with respect to the gage. The magnitudes of the deflections thus obtained were compatible with deflections obtained previously when the pavement was loaded with the Dynaflect and the WES 16-kip vibrator. Strain measurements within the individual layers (Bison coils) were not obtainable because the weights of the tug and the B-727-100 aircraft were insufficient to cause measurable strains in the various layers. Collection of traffic data, environmental data, and temperature-crack width data was initiated at this time by the City of Chicago, Bureau of Engineering.

Additional load-deflection response measurements were made in May 1975. These included pavement deflection measured with the LVDT gages with a plate load device which simulated the load of a B-727 aircraft, and dynamic load-deflection measurements with the WES 16-kip vibrator. Material sampling was accomplished at the sites where the LVDT gages were located and pavement condition surveys were made.

An analysis of all the data collected, both from the earlier evaluation of existing CRC pavements and from the load-deflection tests on runway 4R-22L, was accomplished by ARE. The analyses performed included comparisons of the measured pavement load-deflection response with predicted response (Figure 4-2), analysis of the relationship between crack spacing and crack width, and an analysis of the performance of the pavement. The performance analysis was limited because, since construction of the pavements in 1971, they have experienced low usage and showed no signs of structural deterioration.

Results from the entire study were drawn together and an implementable design procedure for CRC airport pavements and overlays developed. This procedure is compatible with procedures currently used for the design of plain and reinforced jointed pavements. Guidance for handling construction problems that are unique to CRC pavements was formulated.

Modifications to the procedures developed by ARE were made as indicated appropriate by the O'Hare study, and as needed to preclude requirements for estimating traffic, characterizing materials, and computing thickness requirements which are different from those used for other types of pavement. The procedure, as illustrated by the flow chart in Figure 4-3, includes recommended methods for selecting design parameters (traffic estimates and material characteristics), methods for determining slab thickness, methods for determining amount and size of reinforcing steel, details for terminal treatment systems, and details for required construction joints.

As shown by Figure 4-3, the site investigation is identified with those used for other types of pavement. The procedure for selection

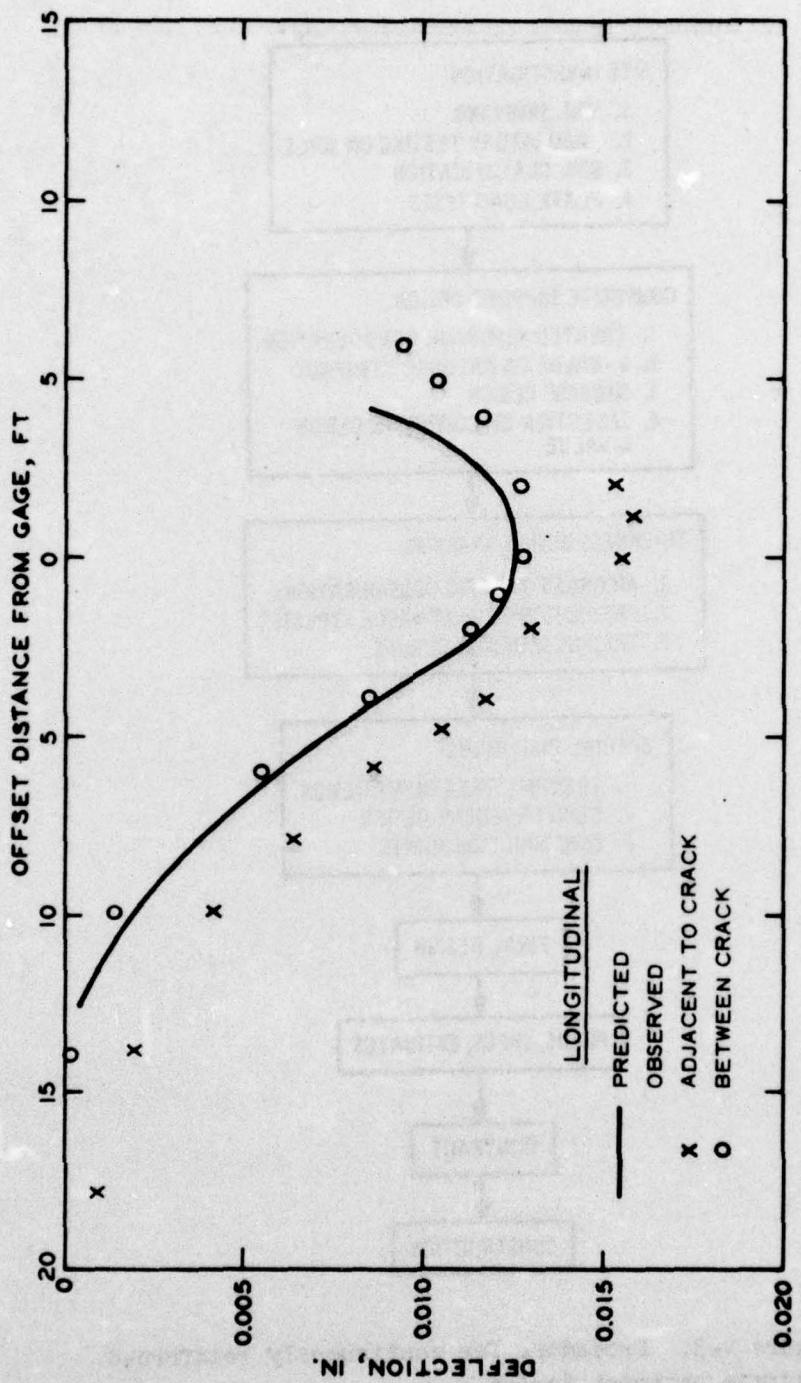


Figure 4-2. Comparison of predicted and observed deflections for the B-727 load at Site 4.

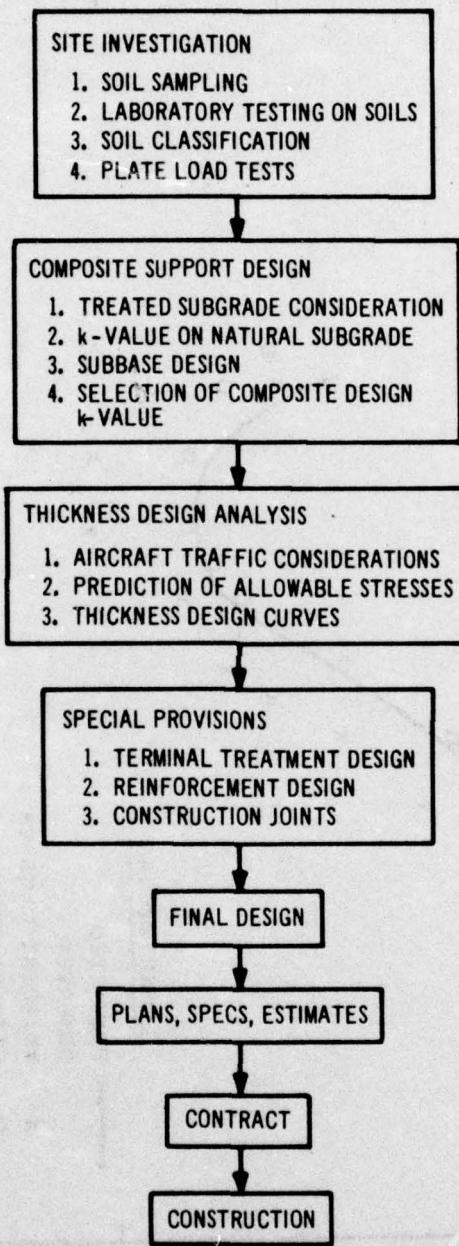


Figure 4-3. Procedure for continuously reinforced concrete pavement design

of a composite support value provides a method for evaluating the increase in support provided by subbase and treated subgrade layers. The methods for considering traffic are commensurate with FAA procedures for selecting the critical design aircraft and relating all other traffic to equivalent traffic with the critical aircraft. The methods for selecting the design thickness have the same basis as current methods, but the charts and nomographs are of different format than design charts used for plain and reinforced jointed pavement. However, they are rather simple, straightforward, and should present no problems in use. It is felt that presentation in the format offers the designer more flexibility should he choose to consider variations in the design parameters. The provisions for design of end anchorage systems, reinforcing steel and construction joints and construction guidance are basically the same as those developed by ARE. Included are recommended details which are unique for CRC; i.e., details for laps at splices in reinforcing bars, details for transverse construction joints, steel placement, end anchorage systems, etc. Procedures are provided for determination of the amount of both longitudinal and transverse steel.

#### 4.6 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

CRC offers the designer an additional alternative to consider when designing a pavement system. The procedures presented, although based on limited experience with airport CRC pavements, provide a means for designing and constructing CRC airport pavements and overlays that will be adequately designed to prevent load-induced cracking during the pavement design life. This is the type distress which is most detrimental to the structural performance of the pavement. The user should note that the procedures do not consider, nor make allowances for, load-induced spalling or distress due to environmental factors or construction conditions. These types of distress are usually not catastrophic, in the sense that the pavement becomes structurally unsound, but the distress may require maintenance and minor rehabilitation to maintain its functional adequacy.

It is recommended that the use of CRC not be postponed until all the answers are available, because many of the problems can only be solved through experience. The following three areas are recommended for further study in order to improve the reliability of the design of CRC pavements and overlays:

- a. Collect additional data for improving the performance criteria.
- b. Identification and description of additional failure modes (in addition to cracking of the slab), and determination of the feasibility of developing design criteria to account for these failure modes.
- c. Identification and quantification of the effects of environmental factors (temperature and moisture regimes) and construction conditions on the performance of CRC.

## CHAPTER 5: FROST AND PERMAFROST

### 5.1 APPLICABLE PUBLICATIONS

This chapter summarizes "Design of Civil Airfield Pavements for Seasonal Frost and Permafrost Conditions," Report No. FAA-RD-74-30, October 1974.

### 5.2 BACKGROUND

FAA criteria for the design of pavements to control the stress caused by differential heaving or foundation weakening due to seasonal frost are based upon concepts developed for aircraft significantly different from present-day commercial aircraft. These criteria do not provide alternatives based upon degree of roughness, proposed usage, or funding available; they often result in unsatisfactory performance or excessive life-cycle cost of pavements and are difficult to enforce. In addition, these criteria do not recognize parameters known to affect the depth of frost penetration and thus do not account for the detrimental effects of seasonal frost conditions.

Presently, the FAA provides no criteria or guidance applicable to the design and construction of pavements in areas of permafrost. This is an area in which the existing technology needs further development; however, some information of value is available and is presented herein for incorporation into FAA design criteria.

### 5.3 PURPOSE AND SCOPE

This study was to:

- a. Delineate the frost susceptibility of various soil groups.
- b. Provide a methodology for the determination of frost penetration depths.
- c. Develop methods of design based on various levels of frost protection.
- d. Prepare design curves for flexible and rigid pavements for the various levels of frost protection.
- e. Provide guidance regarding construction control, new materials, and construction techniques to reduce the detrimental effects of frost on pavement performance.

- f. Present appropriate testing procedures necessary for use of the design methods.
- g. Provide guidance for design of pavements in permafrost areas.

#### 5.4 SUMMARY OF WORK ACCOMPLISHED

All available data sources were used in this study to provide the best technical solution to problems in the design of civil airport pavements for frost and permafrost. The effects of frost action on various soil groups were studied along with the effects of frost heaving and soil weakening after thawing on pavement roughness and cracking. Criteria were established to delineate the degree of frost susceptibility of various soils.

Procedures for determining frost and thaw penetration depths were reviewed along with use of freezing and thawing indexes and a modified form of the Berggren equation. There are four procedures for designing pavements for seasonal frost:

- a. Complete protection.
- b. Limited subgrade frost penetration.
- c. Reduced subgrade strength.
- d. Reduced subgrade frost penetration.

Pavement design guidance for permafrost regions was developed that considers not only the effects of seasonal thawing and freezing cycles but also the effects of construction on the existing thermal balance. Special treatments for bases and subgrades in seasonal frost and permafrost areas were also considered in terms of stabilization and insulating layers.

In addition, field control of pavement construction in areas of seasonal freezing, design of base course drainage, laboratory frost-susceptibility tests, results of freezing tests on natural soils, and thermal models for computing frost and thaw depths were also considered in this study.

#### 5.5 SUMMARY OF FINDINGS AND CONCLUSIONS

This study resulted in the formulation of criteria for use in airport pavement design in seasonal frost and permafrost regions.

Methodology for determining the frost susceptibility of various soil groups and the depth of frost penetration was developed. Four methods of design based on different levels of frost protection and design for permafrost were also developed. In addition, construction control and appropriate testing procedures were established to reduce the detrimental effects of frost on pavement performance.

## CHAPTER 6: AIRCRAFT DISTRIBUTION ON PAVEMENTS

### 6.1 APPLICABLE PUBLICATIONS

This chapter is a summary of "Field Survey and Analysis of Aircraft Distribution on Airport Pavements," Report No. FAA-RD-74-36, February 1975.

### 6.2 BACKGROUND

One of the parameters included in the current FAA design criteria is the number of load repetitions that the pavement will receive during its design life. In the development of the design criteria, it was recognized that the incremental detriment to a pavement, caused by a particular wheel at a particular location, is influenced by factors such as the number of wheels, wheel configuration, load on each wheel, tire contact area, and location of the aircraft on the pavement. To account for the collective influence of these factors in the evaluation of airport pavements, the concept of coverages was introduced. The definition of a coverage differs for flexible and rigid pavements. For flexible pavements, a coverage occurs when each point of the pavement within the traffic lane has been subjected to a tire contact point; for rigid pavements, a coverage occurs when each point of the pavement within the limits of the traffic lane has been subjected to a maximum stress.

The current procedure for converting aircraft passes to coverages (P/C ratio) employs a traffic width which is based upon information developed several years ago by the Corps of Engineers when aircraft traffic flow and volume were considerably different than at present. Current and projected traffic conditions required that this procedure be verified or modified, as a function of the type of runway and the primary using aircraft, by the use of lateral and longitudinal traffic distribution measurements.

### 6.3 PURPOSE

The purpose of this study was to develop a simple and effective method of predicting the lateral and longitudinal traffic distributions

of aircraft on civil airport runways and taxiways. The study included consideration of particular aircraft types from among those commonly used for commercial air transportation and for which field data were collected and analyzed for specific airports selected to represent reasonable variations in the operating environment. Data on aircraft ground speed and weather conditions correlated with observed measured lateral and longitudinal distributions were also obtained and documented.

#### 6.4 SCOPE

The work under this study was accomplished in three phases: data collection, data analysis, and report preparation.

The data collection phase consisted of the development, assembly, and testing of the necessary instrumentation system for:

- a. Determining the lateral and longitudinal distribution of aircraft on runways.
- b. Determining the lateral distribution of aircraft at particular points on taxiways.
- c. Determining aircraft ground speeds on runways and runway exits.
- d. Automatically identifying aircraft by type or recording manually observed aircraft identities.

The remainder of this phase consisted of the use of the instrumentation system to collect pertinent data at nine selected airports. Data which might affect the distribution of the traffic were also obtained and documented. These data included description of displaced thresholds, obstructions, abnormal operational conditions, signal and instrument landing aids, pavement widths, weather, temperatures, altitudes, time of day, etc.

The data analysis consisted of a statistical analysis of the field data for the purpose of identifying and isolating those factors which appear to influence or characterize the distribution of aircraft traffic on runways and taxiways. The current FAA method for determining aircraft traffic distribution was examined by comparisons with the results of the statistical analysis of the field data.

## 6.5 SUMMARY OF WORK ACCOMPLISHED

A system of infrared photoelectric sensors was installed in N-shaped and I-shaped arrays on runways and taxiways at selected civil airports to determine the aircraft velocity, lateral location, and longitudinal location during takeoff, landing, and taxiing operations. Figure 6-1 contains a schematic diagram of the instrumentation system and a typical layout of the system used during the testing operations. Combinations of these basic arrangements were installed along the runway in a manner similar to that of the typical layout in Figure 6-1. All data were recorded on magnetic tape. The data included the date, the time of day, system parameters, mode (takeoff, landing, or taxiing), results of calculations (position and velocity), and other pertinent data.

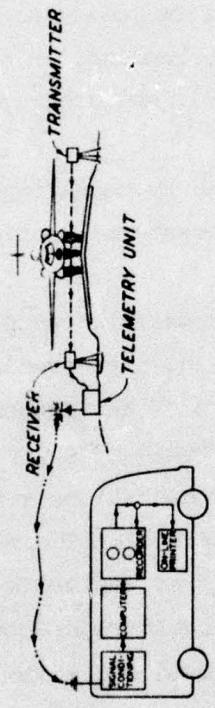
Aircraft identification was accomplished by measuring the aircraft "footprint," since each aircraft of interest had a unique combination of wheelbase, tread, and configuration.

The data collection system accuracy was verified by photographic techniques. Two synchronized 100-mm, high-speed, high-precision cameras were used to obtain sequenced overlapping pairs of photographs of aircraft passing through an N-array. The photographic method proved to be accurate to at least 0.3 ft. Several aircraft operations were recorded by both the photographic equipment and the data collection system. The maximum difference between the offset distances determined by the data collection system and those determined by the photographic method was 0.3 ft. Therefore, it was concluded that the data collection system was adequate for the field survey.

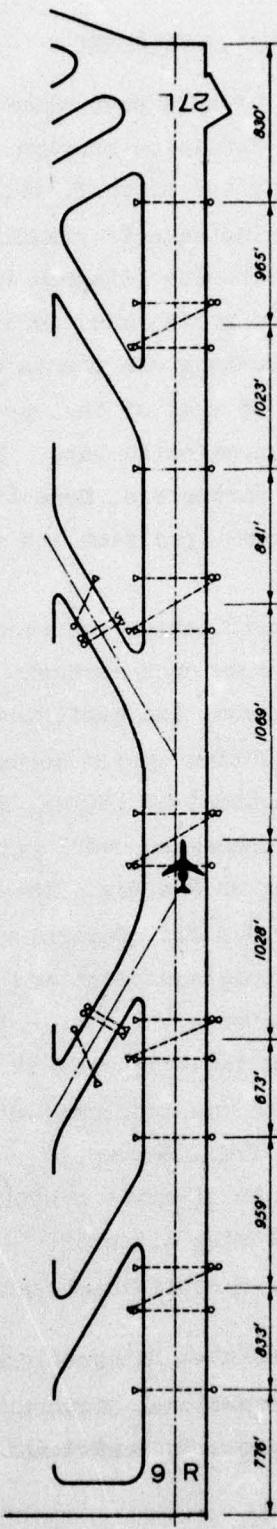
Data relative to aircraft traffic distribution and speed were collected at the following airports:

- a. William B. Hartsfield-Atlanta International Airport, Atlanta, Ga.
- b. Denver-Stapleton International Airport, Denver, Colo.
- c. Miami International Airport, Miami, Fla.
- d. Seattle-Tacoma International Airport, Seattle, Wash.

## TRAFFIC DISTRIBUTION STUDY



SCHEMATIC OF INSTRUMENTATION SYSTEM



TYPICAL LAYOUT OF LIGHT BEAM ARRAYS

Figure 6-1. Schematic of instrumentation system and typical layout of light beam arrays

- e. Greater Buffalo International Airport, Buffalo, N. Y.
- f. Cleveland-Hopkins International Airport, Cleveland, Ohio.
- g. New Orleans International Airport (Moisant Field), New Orleans, La.
- h. Chicago-O'Hare International Airport, Chicago, Ill.
- i. Dallas-Fort Worth Regional Airport, Tex.

These airports were selected as test sites on the basis of their individual or collective potential for providing the desired data, with reasonable variations in operating conditions.

Data were obtained for the following aircraft types: B-707, B-727, B-737, DC-8, DC-9, C-580, B-720, B-747, DC-10, L-1011, C-880, BAC-111, and YS-11. Not all of the above aircraft types operate at each of the selected airports, and some types operate at relatively low frequencies. Besides the consideration of variations in airport altitudes, temperatures, and climatological conditions, the airports were selected to provide in reasonably minimum time as much data as practicable on all of the above aircraft types in general and the first six types in particular.

Lateral aircraft traffic distributions were determined at the following locations of each test site:

a. Runways.

(1) For landings:

- (a) At the approximate point of touchdown or at a point reasonably close thereto.
- (b) At a point a reasonable distance beyond the point of touchdown and prior to the start of exit from the runway.
- (c) At a point close to the exit from the runway.

(2) For takeoffs:

- (a) At a point near the start of takeoff roll.
- (b) At the approximate point of rotation or at a point reasonably close thereto.
- (c) At a point approximately midway between (a) and (b).

b. Taxiways.

- (1) At the exit point on a high-speed or flat-angled exit taxiway from a runway.

(2) At the exit point on a right-angled exit taxiway from a runway.

(3) At a point on the straightaway portion of a taxiway.

Field measurements were collected for the longitudinal distribution of aircraft traffic on runways for the points of touchdown and rotation.

Aircraft ground speeds were determined at the specified runway observation points for lateral distribution and at the exit point on high-speed exit taxiways.

Data were collected for daytime and nighttime hours of operation at each airport; however, the major portion of the data was obtained for daytime operations.

The following aircraft traffic distribution data were collected for each aircraft during the field tests:

- a. Identification of landing, departing, or taxiing aircraft.
- b. Longitudinal location of the point of touchdown relative to the landing end of the runway or displaced landing threshold for landing aircraft.
- c. Longitudinal location of the point of rotation relative to the takeoff end of the runway for takeoff aircraft.
- d. Lateral position relative to the runway center line at the various locations described previously for both landing and takeoff aircraft.
- e. Lateral position relative to the taxiway center line at the previously specified taxiway points for taxiing aircraft.
- f. Ground speeds at those points for which lateral traffic distribution data were collected for landing, departing, and taxiing aircraft.
- g. Date and time of day for each event.

Data on prevailing weather conditions during the collection of field data were obtained and recorded each hour or whenever significant changes occurred. Data on other conditions which might affect the traffic distribution characteristics were also observed and recorded.

One runway at each of the nine airports was instrumented such that data could be collected for both operating directions on the runway. Two of the runways were 200 ft wide; the others were 150 ft wide.

A total of 4359 takeoffs and 5200 landings were recorded during the field survey. Eight high-speed exits were instrumented in the study and a total of 697 operations at these exits were recorded. Seven locations on straight taxiways were instrumented in the study. One location was on a 100-ft-wide taxiway; the others were on 75-ft-wide taxiways and a total of 590 operations at these locations were recorded.

Procedures based on generally accepted methods of statistical analysis were used to analyze the field survey data. The mean and standard deviations of the lateral distributions and ground operating speeds were calculated for each sample--individual or combined aircraft types at individual or combined airports--and were the two primary statistical parameters used in describing and comparing samples. Frequency distributions (histograms) of the observed aircraft to center line offsets were plotted, in terms of the proportionate occurrences in 2-ft intervals on either side of the pavement center line or guideline markings.

It was obvious from inspection of the histograms, such as the one shown in Figure 6-2, that the lateral distribution of aircraft traffic on runways, runway exits, and taxiways would be much more nearly represented by theoretical normal distribution functions, which are currently used to derive pass-per-coverage ratios (Brown and Thompson's report on "Lateral Distribution of Aircraft Traffic," WES Miscellaneous Paper S-73-56, July 1973), rather than by a modified uniform distribution function which had previously been used. This observation was also verified statistically using the chi-square goodness of fit test.

In general, aircraft center-line offsets were (a) to the left of the pavement center-line stripe on runways; (b) to the right of the pavement center-line stripe on straight taxiways; and (c) to the left or right of the guideline on high-speed exits, depending on aircraft operational flow pattern and exit configuration. The computed offset mean on runways, both for landings and takeoffs, generally ranged from 0.5 to 1.5 ft left of the center line on 150-ft-wide pavements and was from 0.8 ft right to 2.5 ft left of the center line on 200-ft-wide pavements. Wide-body and 4-engine aircraft tended to be slightly

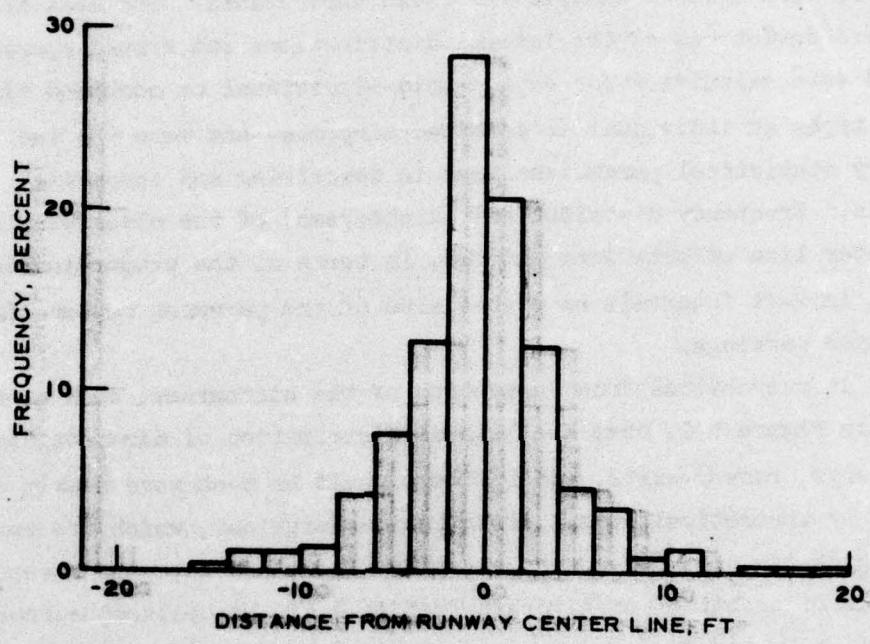


Figure 6-2. Typical histogram of lateral distribution of aircraft traffic on runways during takeoffs (actual data from Cleveland Hopkins International Airport Runway 23L).

farther left than 2- and 3-engine aircraft, but the difference was neither large nor consistent enough to make a distinction among such aircraft groupings.

The shapes of the lateral distribution patterns for takeoffs were generally narrower than those for landings. The computed standard deviations for individual aircraft types, compared at the various airports, varied generally from 3 to 8 ft for takeoffs and from 4 to 9 ft for landings. There was no consistent correlation of the standard deviation with respect to aircraft type or size.

The standard deviations for takeoffs, for all the airports combined, varied among individual aircraft types generally from 6 to 9 ft in the vicinity of liftoff and from 5.5 to 7 ft in earlier portions of takeoff roll. On 200-ft-wide runways, the standard deviations in the vicinity of liftoff were about the same as those in the same area on 150-ft-wide runways but about 1.5 ft wider in earlier portions of takeoff roll. The normal distribution curves for takeoff operations of all aircraft at all airports are shown in Figure 6-3.

The standard deviations for landings, for all the airports combined, varied among individual aircraft types generally from 7 to 8.5 ft, except near the end of landing rolls where, as expected, exit influences were much in evidence. The normal distribution curves for the case of landing operations of all aircraft at all airports are also shown in Figure 6-3.

Factors such as night operations, crosswinds, and wet pavements had an apparent effect on aircraft lateral distributions on runways; however, their overall impact was not considered significant because the effect of any one such factor was (a) relatively small or generally not consistent, (b) infrequent in occurrence, or (c) compensated for or nullified by other factors in the overall operating conditions.

In general, 90 to 95 percent of touchdowns occurred in the first 3000 ft from the threshold, 70 to 85 percent occurred in the first 2000 ft, and 15 to 25 percent occurred in the first 1000 ft. A higher percentage of 2-engine aircraft touchdowns occurred closer to the

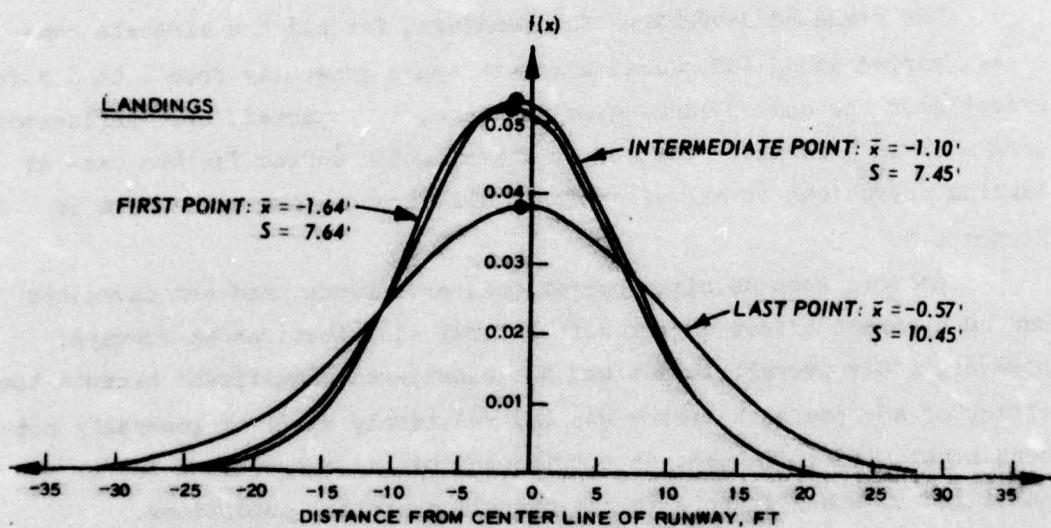
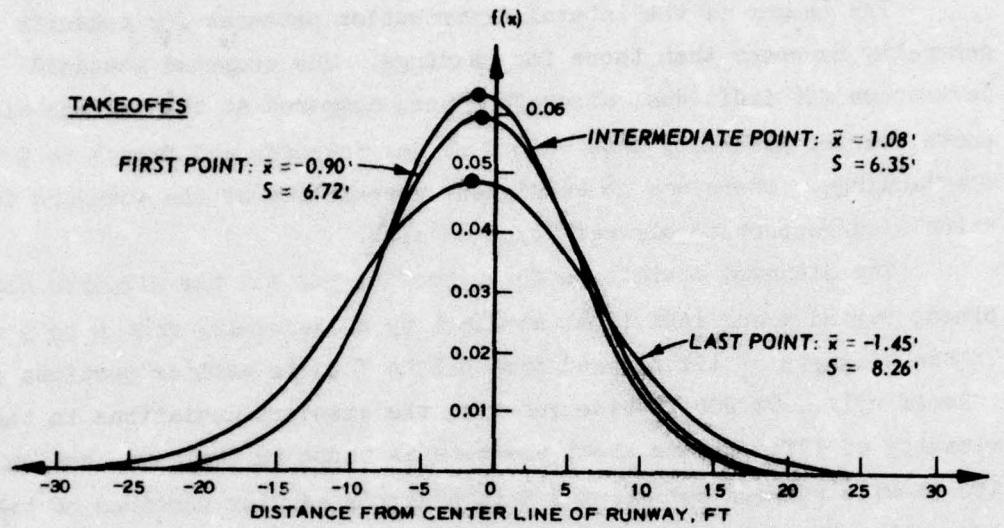


Figure 6-3. Summary of lateral distributions on runways for all aircraft at all airports

threshold in comparison with wide-body and 4- and 3-engine narrow-body aircraft.

The standard deviations of the observed lateral distributions on high-speed exits were generally greater than those on runways and were influenced by aircraft operational flow pattern and exit configuration. The standard deviations ranged approximately from 8 to 10.5 ft, with the upper limit probably more representative of typical exit configurations and their normal usage.

The computed offset mean of the lateral distributions on straight taxiway sections was approximately 2 ft right of the center line on the 75-ft-wide taxiways and approximately 3 ft right of the center line on the 100-ft-wide taxiway. The standard deviations were much smaller than those on runways, ranging approximately from 2.5 to 4 ft on the 75-ft-wide taxiways and averaging about 6 ft on the 100-ft-wide taxiway. The average taxiing speeds ranged between 35 and 45 fps on both the 75- and 100-ft-wide taxiways.

The results of the analysis of the field survey data yield the following conclusions:

- a. The lateral distribution of aircraft on runways, runway exits, and taxiways is much more nearly represented by theoretical normal distribution functions than by uniform distribution functions.
- b. The properties of the aircraft wheel-path distribution are summarized in the tabulation on page 6-12.
- c. The overall impact of such factors as night operations, cross-winds, and wet pavements is not considered to be a significant influence on the distribution of aircraft traffic.

Pavement Type	Pavement Width, ft	Distribution Characteristics	
		Average Offset,* ft	Standard Deviation, ft
Runways (landings)	150	0.9 to 1.5 left	7.0 to 10.3
	200	0.8 right to 2.3 left	9.0 to 11.2
Runways (takeoffs)	150	0.5 to 1.2 left	6.0 to 8.3
	200	2.3 to 2.5 left	7.5 to 8.2
Straight taxiways	75	2.1 right	2.5 to 4.0
	100	3.2 right	6.0
High-speed runway exits	Varies	—**	8.0 to 10.5

\* Aircraft to center line offset, measured relative to pavement center line or guideline markings.

\*\* Average offset was to the left or to the right of the guideline, depending upon the aircraft operational flow pattern and exit configuration.

## CHAPTER 7: NEW CRITERIA FOR PAVEMENT DESIGN AND CONSTRUCTION

### 7.1 APPLICABLE PUBLICATIONS

The work reported herein represents a follow-up to work accomplished under a previous Inter-Agency Agreement and reported in Report No. FAA-RD-70-77, "Airfield Pavement Requirements for Multiple-Wheel Heavy Gear Loads."

### 7.2 BACKGROUND

Under a jointly funded FAA, Army, and Air Force study, full-scale accelerated traffic tests were conducted for the purpose of validating or revising design criteria for flexible and rigid pavements subjected to multiple-wheel heavy gear loads. As a result of these tests, the procedure for developing thickness design criteria for flexible pavements was modified and the design procedure for rigid pavements was validated. It was therefore necessary to develop new thickness design curves for flexible pavements and to extend the rigid pavement design curves to include the new wide-body jets. Modification of the flexible pavement design curves also made it necessary to revise compaction requirements.

### 7.3 PURPOSE

The purpose of this study was to update design criteria for flexible and rigid pavements to be constructed at civil airports.

### 7.4 SCOPE

Design curves and compaction requirements were developed for use in designing flexible and rigid pavements subjected to loads resulting from single-wheel, dual-wheel, and dual-tandem gears or from the B-727, DC-10-10, DC-10-30, L-1011, and Concorde aircraft. Characteristics of the landing gears and aircraft used to develop the criteria are shown in Table 7-1.

Table 7-1  
Aircraft Characteristics

Aircraft	Main Gear Type	Contact Area sq. in.	Tire Pressure psi	Gross Weight kips	Departure Range	Tire Spacing in.
--	Single-wheel	190	75	30	1,200 to 25,000	--
		238	90	45	1,200 to 25,000	--
		271	105	60	1,200 to 25,000	--
		297	120	75	1,200 to 25,000	--
--	Dual-wheel	148	80	50	1,200 to 25,000	20
		162	110	75	1,200 to 25,000	21
		170	140	100	1,200 to 25,000	23
		223	160	150	1,200 to 25,000	26
		238	200	200	1,200 to 25,000	30
		99	120	100	1,200 to 25,000	20 by 45
		127	140	150	1,200 to 25,000	20 by 45
B-747	Dual-tandem (4)	148	160	200	1,200 to 25,000	21 by 46
		198	180	300	1,200 to 25,000	26 by 51
		238	200	400	1,200 to 25,000	30 by 55
		245	97	400	1,200 to 25,000	44 by 58
		245	121	500	1,200 to 25,000	44 by 58
		245	145	600	1,200 to 25,000	44 by 58
		245	169	700	1,200 to 25,000	44 by 58
DC-10-10	Dual-tandem	193	193	800	1,200 to 25,000	44 by 58
		218	218	900	1,200 to 25,000	44 by 58
		294	81	200	1,200 to 25,000	54 by 64
		294	121	300	1,200 to 25,000	54 by 64
		294	162	400	1,200 to 25,000	54 by 64
		294	202	500	1,200 to 25,000	54 by 64
		294	242	600	1,200 to 25,000	54 by 64

(Continued)

Table 7-1 (Concluded)

Aircraft	Main Gear Type	Contact Area sq in.	Tire Pressure psi	Gross Weight kips	Departure Range	Tire Spacing in.
DC-10-30	Dual-tandem and dual center-line	331	57	200	1,200 to 25,000	54 by 64*
		331	86	300	1,200 to 25,000	54 by 64*
		331	115	400	1,200 to 25,000	54 by 64*
		331	143	500	1,200 to 25,000	54 by 64*
		331	172	600	1,200 to 25,000	54 by 64*
		331	201	700	1,200 to 25,000	54 by 64*
L-1011	Dual-tandem	282	84	200	1,200 to 25,000	52 by 70
		282	126	300	1,200 to 25,000	52 by 70
		282	168	400	1,200 to 25,000	52 by 70
		282	211	500	1,200 to 25,000	52 by 70
		282	253	600	1,200 to 25,000	52 by 70
		282	295	700	1,200 to 25,000	52 by 70
Concorde	Dual-tandem	247	96	200	1,200 to 25,000	26.7 by 65.7
		247	144	300	1,200 to 25,000	26.7 by 65.7
		247	192	400	1,200 to 25,000	26.7 by 65.7
		247	240	500	1,200 to 25,000	26.7 by 65.7
		247	288	600	1,200 to 25,000	26.7 by 65.7

\* Center-line gear spacing is 37.5 in.

## 7.5 SUMMARY OF WORK ACCOMPLISHED

Thickness design curves for the above aircraft were developed for flexible pavements using conventional-type construction. These curves relate soil strength expressed as CBR, aircraft gross weight in pounds, traffic in terms of annual departures, and thickness in inches. Typical flexible pavement design curves are shown in Figure 7-1. To use the criteria, the designer must enter the curves with soil strength and move vertically to the design gross aircraft weight, horizontally to the annual departure level, and then vertically to the thickness requirement. This thickness represents the thickness of material required above a soil layer having the strength with which the curves were entered.

Also developed were minimum base course requirements for flexible pavements, which are shown in Figure 7-2. This figure is entered with the total thickness of pavement above the subgrade and with the CBR of the subgrade to determine the minimum thickness of base course.

Typical compaction requirements curves for flexible pavements are shown in Figure 7-3. This figure is entered with the percent compaction, and the depth at which this percent compaction is required is obtained.

Thickness design curves relating concrete flexural strength, soil strength, load, traffic, and thickness were developed. Typical rigid pavement design curves are shown in Figure 7-4. To use the criteria, the designer enters the curves with the flexural strength and moves horizontally to the modulus of soil reaction  $k$ , vertically to the design aircraft gross weight, horizontally to the annual departures, and then vertically to the pavement thickness. This is the thickness of pavement required above the subgrade or base. A description of the procedures used to develop the flexible and rigid pavement design curves is presented below.

## 7.6 DEVELOPMENT OF DESIGN CURVES

Design curves were prepared for use in designing both flexible and rigid airport pavements. These curves relate the parameters necessary to adequately design a pavement for a 20-year life.

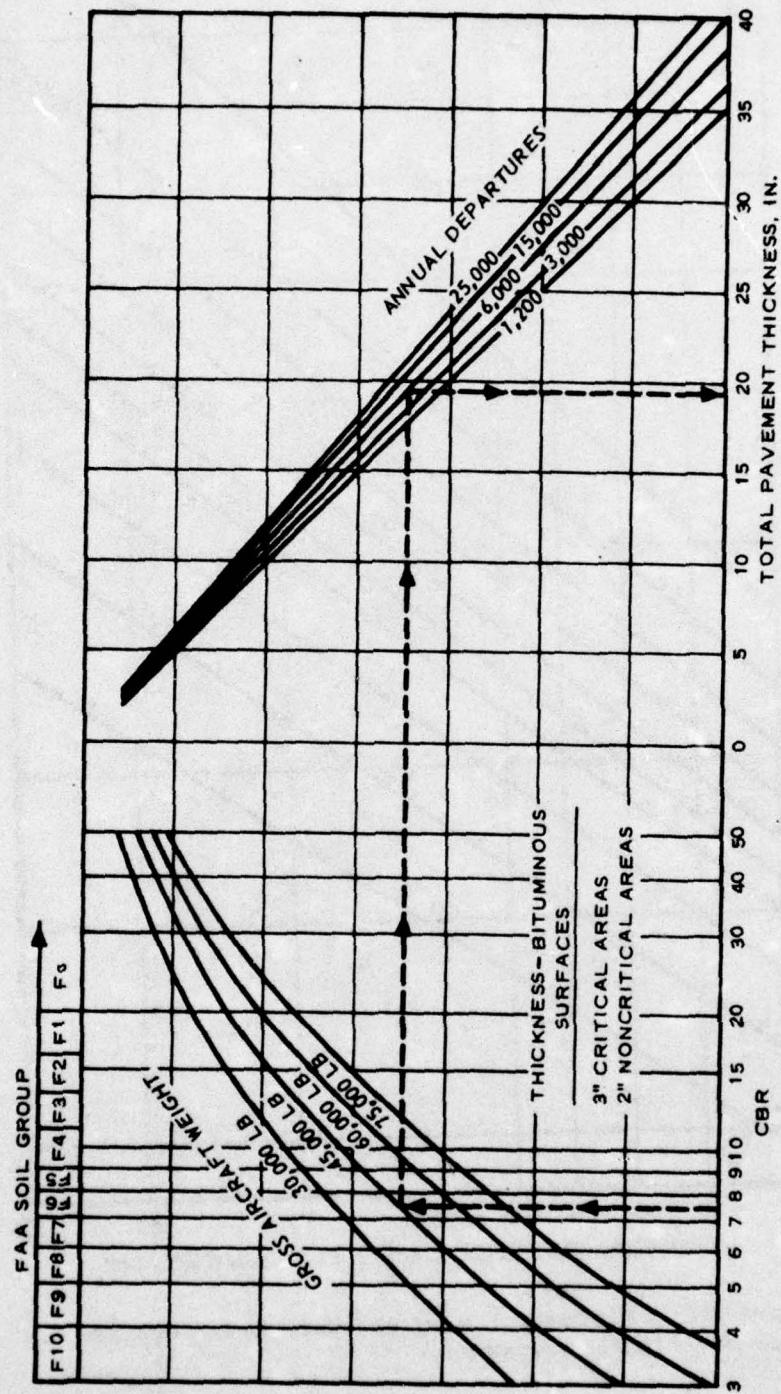


Figure 7-1. Typical flexible pavement design curves for critical areas

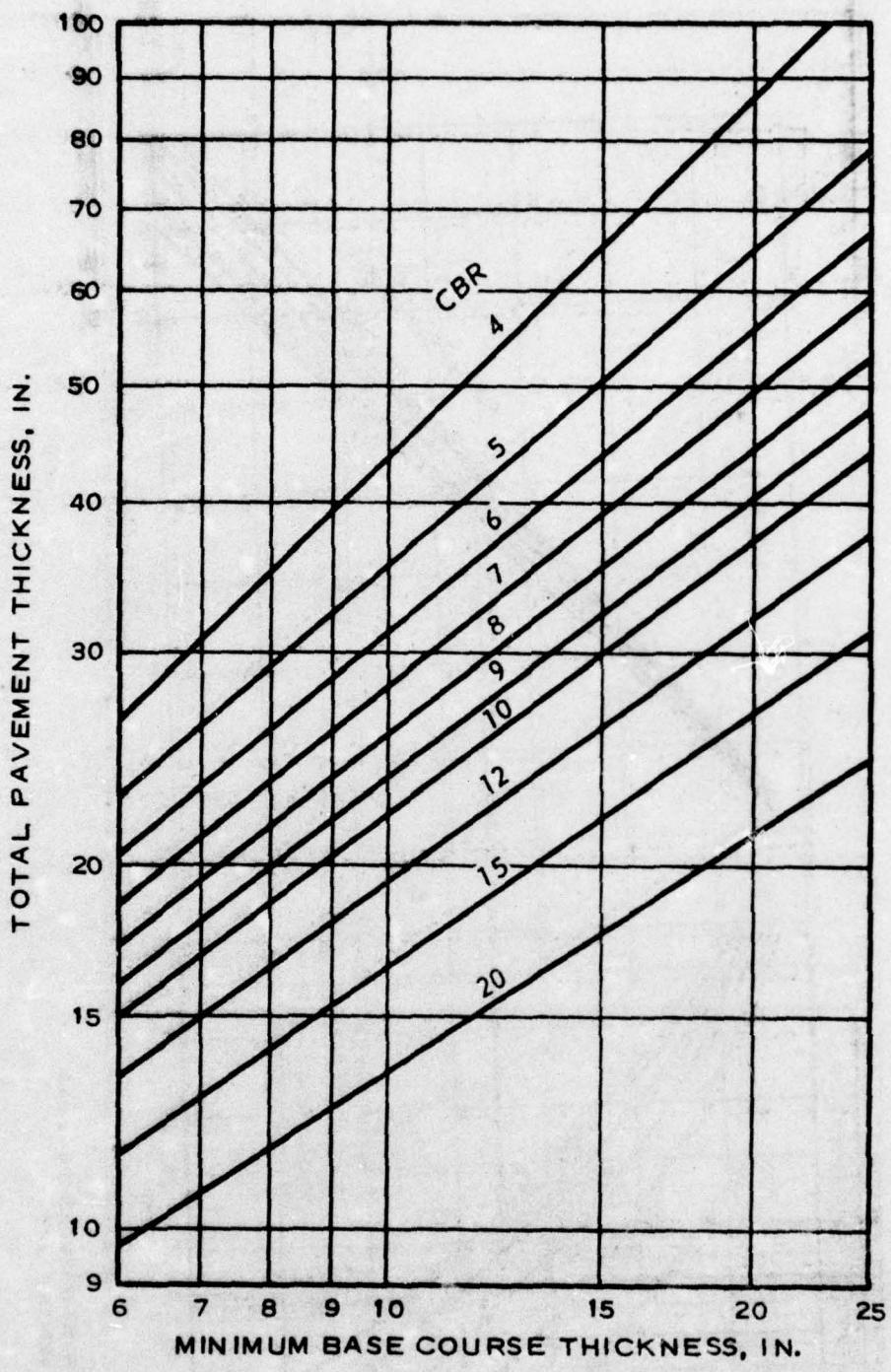


Figure 7-2. Minimum base course thickness requirements for flexible pavement

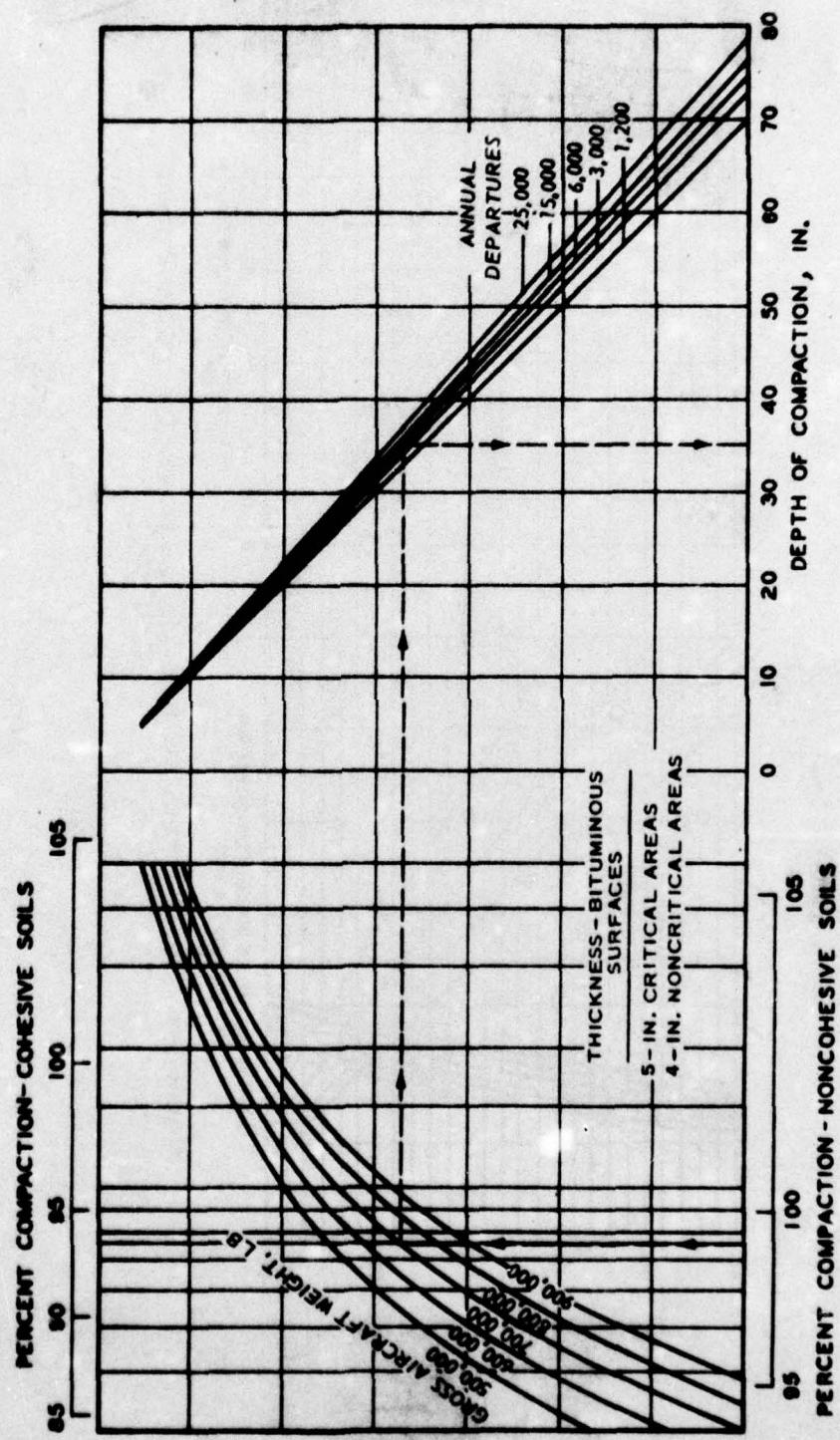


Figure 7-3. Typical flexible pavement compaction requirements curves for critical areas

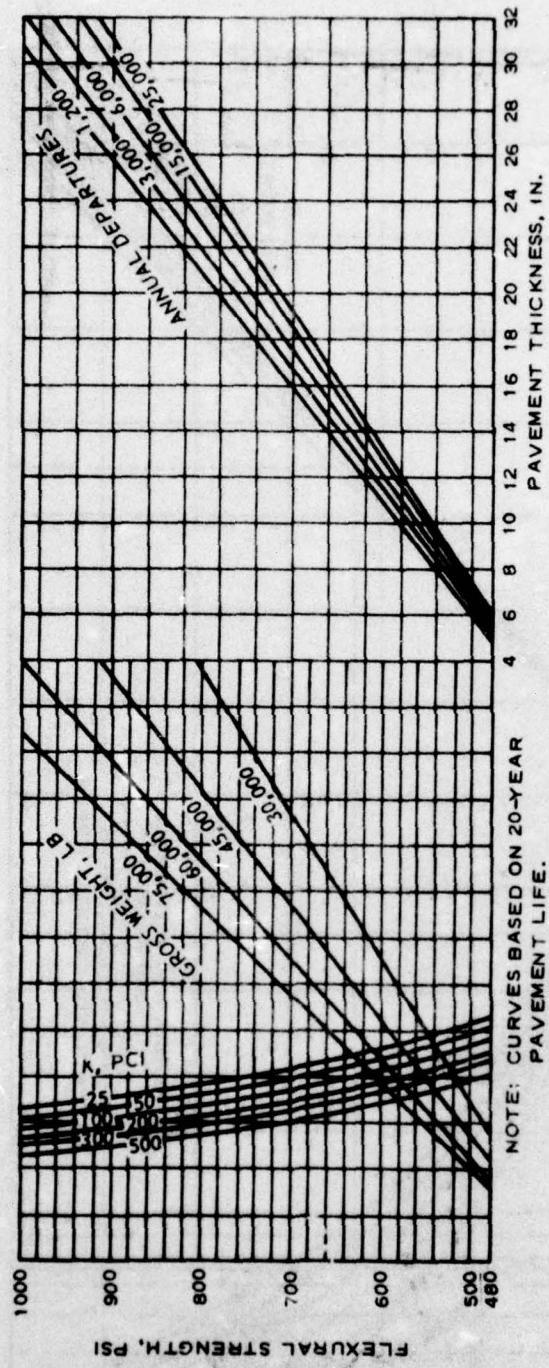


Figure 7-4. Typical rigid pavement design curves for critical areas

### 7.6.1 FLEXIBLE PAVEMENT

Development of the flexible pavement design curves consisted primarily of determining the equivalent single-wheel load (ESWL) for an aircraft and then determining the thickness required to protect a layer of soil from failure due to repetitions of that ESWL.

The ESWL is based on the ratio of maximum deflections beneath a multiple-wheel group and one wheel of that group, computed assuming a homogeneous, isotropic, half-space loaded by uniformly distributed circular loads. The ESWL varies with depth and is determined at pertinent depths or at sufficient depths to form a curve of ESWL versus depth. An example of an ESWL determination can be found in WES Instruction Report No. 4, "Developing a Set of CBR Design Curves." The methodology has been computerized for treatment of complex landing gear geometries and the program is available. The procedure is first applied using all wheels on the main gears. This approach generally results in maximum thickness requirements for a specific aircraft. When it is found that some combination of wheels (i.e., wheels on one main gear) results in greater thickness requirements, then that combination of wheels is used to determine the ESWL.

Once the ESWL has been determined, the next step is to calculate the thickness of superior (stronger) material required above a layer of soil of known strength to prevent shear failure within this layer of soil. The following equation is used to determine this thickness:

$$t = \alpha \sqrt{A} \left[ -0.0481 - 1.1562 \log \left( \frac{CBR}{P_e} \right) - 0.6414 \log \left( \frac{CBR}{P_e} \right)^2 - 0.4730 \log \left( \frac{CBR}{P_e} \right)^3 \right] \quad (7-1)$$

where

$t$  = thickness of the pavement structure above a soil layer of given strength, in.

$\alpha$  = load repetition factor which varies with the amount of traffic as shown in Figure 7-5, in which coverages are the measure of traffic volume. Coverages are related to aircraft departures and are determined by dividing the number of departures by a pass-per-coverage ratio, where a departure is considered one pass. The pass-per-coverage ratio is determined as described in WES Miscellaneous Paper S-72-56, "Lateral Distribution of Aircraft Traffic," and is dependent upon the tire contact area and number and arrangement of tires on the aircraft. Pass-per-coverage ratios for various aircraft are listed in Table 7-2

A = contact area of one tire on the landing gear, sq in.

CBR = strength of the soil (determined by Method 101 of Military Standard MIL-STD-621A)

$p_e$  = ESWL or single-wheel load (SWL) tire pressure, psi. For multiple-wheel gears,  $p_e = ESWL/A$ , and for single-wheel gears,  $p_e = SWL/A$ . This is an artificial tire pressure for multiple-wheel loads that is consistent with use of the contact area of one tire and has no relation to actual tire inflation pressure. However, for SWL, this pressure is the actual average contact pressure and is nominally the same as the tire inflation pressure

Table 7-2

Pass-per-Coverage Ratios for  
Various Aircraft

Aircraft	Pass-per-Coverage Ratio
B-727-100	3.25
B-723-200	3.25
DC-9-30	3.58
B-707-100	1.62*
DC-8-10	1.57*
DC-10-10	1.82*
DC-10-30	1.71
C-880	1.84*
B-747	1.85*
L-1011	1.81*
Concorde	1.52*

\* Pass-per-coverage ratio for rigid pavement for this aircraft is twice the value shown.

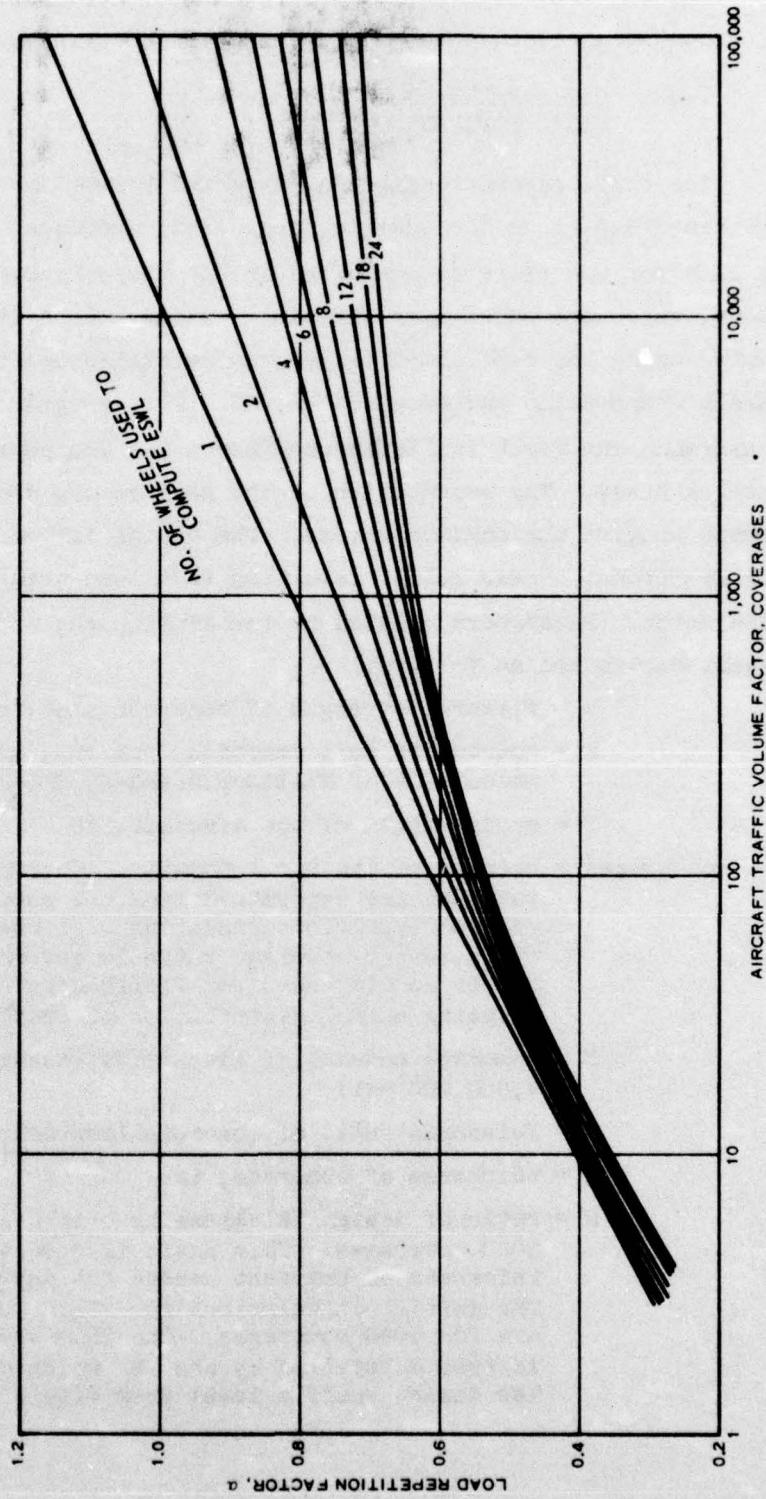


Figure 7-5. Load repetition factor versus aircraft traffic volume factor

The computed thicknesses are plotted on nomographs relating CBR, gross weight, and annual departures as shown in Figure 7-1.

#### 7.6.2 DEVELOPMENT OF RIGID PAVEMENT CRITERIA

The rigid pavement criteria presented herein are based upon the Westergaard equations for edge loading. This approach is a departure from that for the criteria contained in FAA Advisory Circular AC 150/5320-6A, which are based upon interior loading. Concrete stresses were computed using the H-51 computer program developed by the General Dynamics Corporation and extended by WES. This program is based upon the approach set forth in "Influence Charts for Concrete Pavement," by Pickett and Ray. The orientation of the gear on the slab can be selected to give the maximum stress. The design curves were developed using 25 percent stress relief resulting from load transfer devices in the joint. Parameters related to the development of the rigid pavement curves are as follows:

R = flexural strength of concrete slab, psi

k = modulus of soil reaction, psi/in., as measured by Method 104 of Military Standard MIL-STD-621A

P = gross weight of the aircraft, lb

coverages = measure of traffic intensity. Coverages for a rigid pavement are determined from the number of maximum stress repetitions resulting from the aircraft passes. The pass-per-coverage ratio is determined in the same manner as discussed for flexible pavements (i.e., assuming normal distribution of traffic)

E = concrete modulus of elasticity (assumed to be 4,000,000 psi)

$\mu$  = Poisson's ratio of concrete (assumed to be 0.15)

h = thickness of concrete, in.

H = ratio of design thickness to thickness needed for 5000 coverages. This ratio is used to determine the thickness of pavement needed for any coverage level. The initial calculations for rigid pavement curves are for 5000 coverages. The 5000 coverage thickness is then multiplied by the H which corresponds to the design traffic level from Figure 7-6

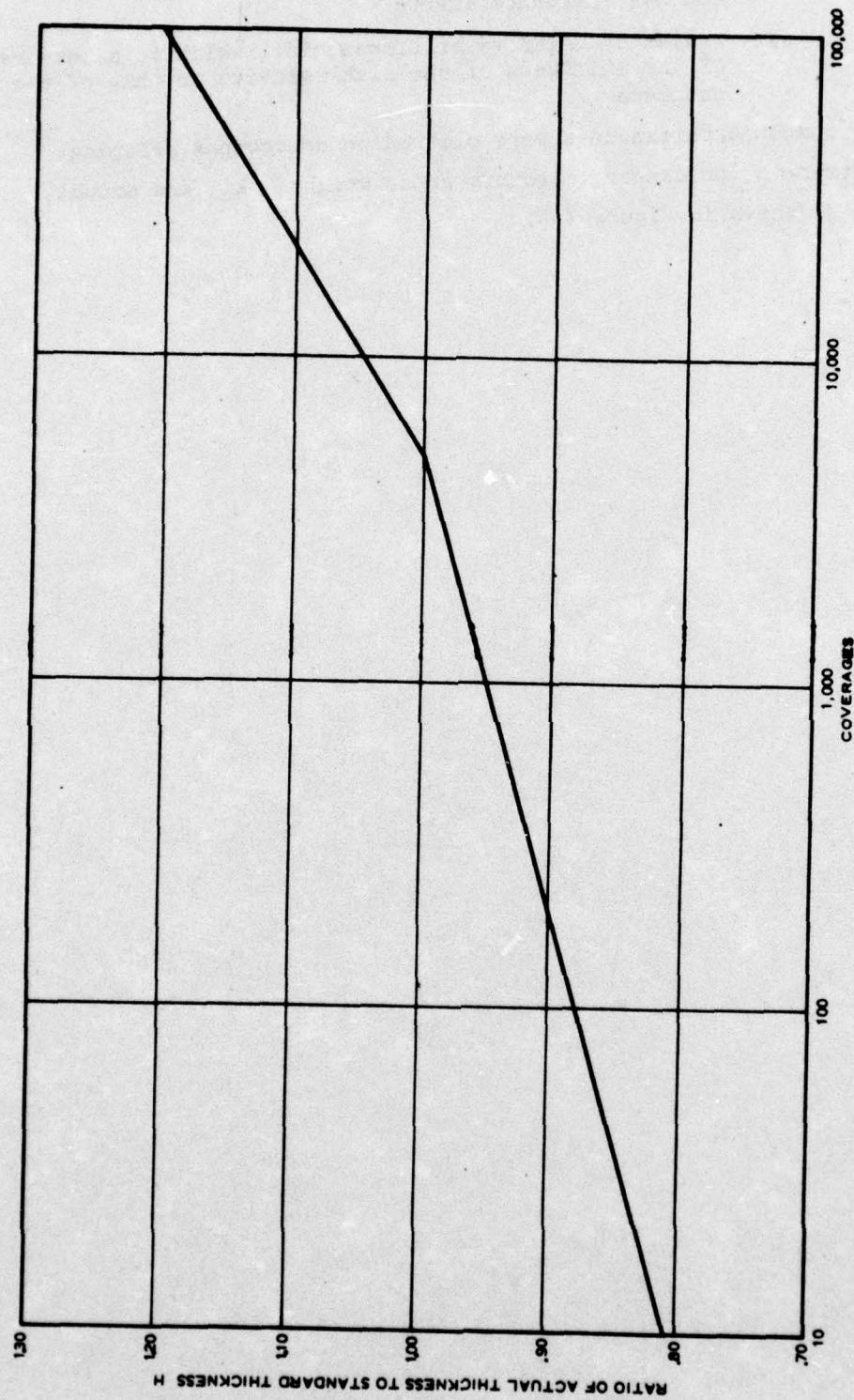


Figure 7-6.  $H$  ratio versus coverages

$\sigma_e$  = maximum edge stress as determined from the Pickett and Ray influence charts

$i$  = radius of relative stiffness, in., which is a measure of the stiffness of the slab relative to that of the subgrade

The computed thicknesses were plotted on nomographs relating flexural strength, thickness, aircraft gross weight,  $k$ , and annual departures as shown in Figure 7-4.

## CHAPTER 8: POROUS FRICTION SURFACE COURSES

### 8.1 APPLICABLE PUBLICATIONS

This chapter summarizes the following two reports: "Porous Friction Surface Course, Interim Report," Report No. FAA-RD-73-197, February 1975, and "Field Performance of Porous Friction Surface Course," Report No. FAA-RD-74-38, April 1976.

### 8.2 BACKGROUND

Aircraft hydroplaning occurs when a film of water separates a high-speed aircraft tire from the pavement causing a loss of directional control and braking. A porous friction course (PFC) is an open-graded bituminous pavement surfacing course designed with a high percentage of internal voids and a coarse macrosurface texture. The high porosity permits a quantity of surface water to drain laterally, internally as well as on the surface, and thus reduce the water film thickness and the hydroplaning potential. The PFC's rough surface texture provides tire-to-aggregate contact above the water film, thereby increasing skid resistance.

### 8.3 PURPOSE

The purpose of this study was to develop laboratory mix design procedures for PFC mixes and to prepare PFC construction specifications.

### 8.4 SCOPE

A three-element program was undertaken as the first phase of the study to determine the applicability of PFC's for airport surfacings. First, a literature survey was conducted to establish a state of the art for PFC mixes. Second, the construction and performance of PFC's were observed at a number of airports and airfields to determine proper construction procedures and pavement characteristics that result in the best performance. Third, a laboratory study was conducted to identify material characteristics (aggregate and bituminous materials) that correlate with field performance and to establish

procedures for designing PFC's. The second phase consisted of additional performance studies and laboratory testing to improve the mix design procedures and construction specifications.

## 8.5 SUMMARY OF WORK ACCOMPLISHED

### 8.5.1. LITERATURE SURVEY

The literature survey established the state of the art in the use of PFC's, the design procedures that had been used, and the agencies that had an interest in such pavements. It was determined through this survey that the aggregate gradations for PFC's had been fairly well established through experience. Therefore, one of these gradations was selected for use in the laboratory studies. In addition, the experience of others reported in the literature was used in planning the laboratory and field phases of this study as well as for guidance in evaluating laboratory results and field observations.

### 8.5.2. LABORATORY STUDIES

A series of tests was outlined to establish the material and mixture requirements for PFC's. Included in this series were determinations of density, shear strength, bitumen drainage, aggregate degradation, permeability, and surface area as well as selection of an acceptable compaction method for PFC specimens. Three aggregate types (a limestone, a chert gravel, and a slag) and three penetration grades of asphalt cement having two viscosities (60-70, AC-20; 85-100, AC-20; and 200-300, AC-5, low on viscosity at 140° F) were selected for preparing PFC specimens to encompass a range of aggregate-asphalt combinations.

Two methods of compacting the PFC specimens were investigated: impact compaction, which is applied according to the Marshall procedure, and gyratory compaction, which is applied using a gyratory compaction machine. Based on data obtained with these two methods, a standard impact compaction procedure was selected consisting of 10 blows of the Marshall hand hammer on one side of the specimen. A standard gyratory compaction procedure was also selected which consisted of 10 revolutions at a 1-deg angle of gyration and a 200-psi ram pressure.

Studies were conducted to determine an acceptable means of measuring the density of PFC mixtures. A physical measuring technique employing the specimen weight in air and its measured dimensions was adopted as the standard method of determining density due to the consistency of data obtained with it.

Studies were conducted to determine if a relationship between direct shear strength and asphalt content could be used in establishing an acceptable PFC mix design. These data indicated that the maximum shear strengths developed by the specimens did not vary greatly over the range of asphalt contents used in the mixtures.

A testing program was conducted to evaluate the effectiveness and reproducibility of bitumen drainage tests for use in specifying the optimum amount of asphalt in a PFC mix. Generally, these tests indicated a range of several percentage points in asphalt content for a given drainage condition over which an acceptable asphalt content could be chosen.

Since toughness and abrasion resistance are important characteristics in identifying the satisfactoriness of materials used in PFC's, Los Angeles abrasion tests and special gyratory degradation tests were used to evaluate the aggregate. These tests produced a relative comparison of aggregate quality through an accelerated test method.

Permeability tests were used to develop a limiting value of asphalt content to achieve acceptable drainage. Analyses of permeability of laboratory PFC specimens and field PFC installations were used to evaluate laboratory compaction efforts. The results of these tests led to the adoption of the standard laboratory compaction effort. Observations and experience gained during the tests led to the selection of a standard size for laboratory specimens to be used for the permeability test.

Centrifuge kerosene equivalency (CKE) tests were investigated as one means of determining an optimum asphalt content. This test involves relating an aggregate surface area constant to the required amount of asphalt. Estimates of asphalt content using this method were found to be within acceptable limits for observed satisfactory field performance.

### 8.5.3. FIELD SURVEYS

A number of airports and airfields were visited to observe the construction and performance of PFC's. Visual inspections were made at each site, field tests were conducted where possible, and core specimens were taken for laboratory evaluation. The visual inspections involved observing the PFC's for surface appearance, cracking, raveling, damage caused by snow removal equipment, and loose aggregate particles. In addition, comments from airport personnel were solicited on PFC performance. Permeability and skid resistance measurements were made on pavements that were accessible.

From data on construction collected in the field surveys the binder temperature-viscosity relation was identified as a primary factor in obtaining satisfactory construction and maintaining the permeability characteristics of PFC's. With a reasonable given volume of binder and a uniform gradation, excessive binder drainage is eliminated and adequate permeability of the PFC maintained.

Field evaluations conducted in 1973, 1974, and 1975 provided significant data by which to evaluate longer term performance of PFC. The field evaluation determined skid resistance, permeability, density, voids, binder content, binder penetration, binder viscosity, aggregate gradation, and pavement condition at the time of the evaluation. These evaluation data were used to validate a PFC design method and recommended specifications. As a result a PFC mixing viscosity and modified 1/2-in. maximum-size aggregate gradation were adopted.

### 8.6 SUMMARY OF FINDINGS AND CONCLUSIONS

PFC's should be considered only for use on structurally sound flexible pavements since cracks in base pavements will reflect through the PFC in a short period of time and will subsequently ravel.

Although there is a tendency for the harder penetration grade asphalt cement (60-70) used as a binder material to stabilize sooner than the softer penetration grade asphalt cement (200-300), the harder penetration grade asphalt cement generally produces lower wet skid resistance

values. An 85-100 penetration grade asphalt cement could be used to counteract these effects. It is believed that the best performance will be obtained from PFC mixes designed using the same types of aggregate and asphalt as used in construction of the base pavement.

Neither laboratory test results nor field observations of PFC mixes showed any correlation between performance and Los Angeles abrasion values that could be used as a basis for changing the Los Angeles abrasion criteria for use in the specification. A lack of difference in performance of PFC pavements attributable to aggregate quality could be the result of all PFC jobs under study being constructed with high-quality aggregate.

A recommended gradation for PFC is shown below:

<u>Sieve Size</u>	<u>Percent Passing by Weight</u>
1/2 in.	100
3/8 in.	80-100
No. 4	25-40
No. 8	12-20
No. 200	3-5

Based on this gradation of aggregate, permeability of PFC mixes, and field performance, it was determined that the minimum thickness of PFC's should be 3/4 in. and the maximum should be 1 in. This thickness of properly designed PFC will provide the necessary porosity. The thickness should be kept to a minimum to reduce any tendency of the PFC to densify under traffic.

The design procedure for PFC mixes is predicated on meeting various material requirements of the asphalt and aggregate. Determination of the asphalt content should be accomplished using the relation  $2K_c + 4.0$ , where  $K_c$  is determined from the California Centrifuge Kerosene Equivalency test. The temperature of mixing should be established using a viscosity-temperature relationship for the job asphalt. Validation of the mix design can include the permeability test. The design procedure for PFC mixes should consist of the following:

- a. Determine  $K_c$  and estimate the asphalt content by the relation  $2K_c + 4.0$ .
- b. Develop the viscosity-temperature relationship for the job asphalt for use in establishing the field mixing temperature and select the mixing temperature at a viscosity of  $275 \pm 25$  centistokes.
- c. If desired, conduct a laboratory permeability test to establish that the PFC mix design will have acceptable permeability.

#### 8.7 NEEDED STUDIES

It has been proposed that a 3/4-in. maximum-size aggregate gradation be used for PFC's. However, some problems with raveling have been observed with this gradation, and it is suggested that a more detailed study of this type of gradation be made. Elastomeric-modified asphalt in the form of neoprene-modified asphalt has performed very well, and a study of other elastomers as asphalt modifiers is suggested.

## CHAPTER 9: STRUCTURAL LAYERS

### 9.1 APPLICABLE PUBLICATIONS

This chapter summarizes portions of Report No. FAA-RD-73-198, "Comparative Performance of Structural Layers in Pavement Systems," Volume I, "Design, Construction, and Behavior Under Traffic of Pavement Test Sections," June 1974, and Volume II (in preparation), "Analysis of Test Section Data and Presentation of Design and Construction Procedures."

### 9.2 BACKGROUND

It is generally recognized and field observations have verified that pavement performance is related to the quality and thickness of layers making up the pavement. Present criteria generally yield minimal thicknesses based upon the use of specified materials (hereinafter referred to as "conventional" materials or conventional pavements) that experience has shown will give satisfactory performance. Experience in accelerated testing of full-scale pavement sections has shown that the use of materials having durability and strengths greater than those of conventional materials will permit a reduction in required thicknesses for equal performance. The quality of the materials can be improved through either selective use of existing materials or through the chemical or bituminous stabilization (hereinafter simply referred to as "stabilization") of existing materials. Stabilization can be used to transform locally available materials of either specified or marginal quality into acceptable or high-quality structural layers, thereby reducing the required thickness of the locally available material. These reductions in thickness requirements often offset the additional expenses involved in stabilization. The performance and possible economic advantages of stabilization have therefore been fairly well defined. The basic problem underlying further advancement of the state of the art is that of developing a method for incorporating the increase in performance into design considerations.

### **9.3 PURPOSE**

The purpose of this study was to develop design criteria and construction procedures for airport pavements incorporating high-quality structural layers.

### **9.4 SCOPE**

The study was accomplished as follows:

- a. Two specially designed pavement test sections composed of items representing a range of stabilization efforts were constructed and trafficked to failure under controlled, accelerated conditions.
- b. Data from the two specially designed test sections and from previous studies of stabilized and high-quality material pavement test sections were compiled and used as input to an analysis of construction and performance characteristics of pavements incorporating high-quality structural layers.
- c. The results of the analysis were used to formulate design criteria and construction procedures.

### **9.5 SUMMARY OF WORK ACCOMPLISHED**

#### **9.5.1 DATA COLLECTION**

Two specially designed test sections, including four items surfaced with asphaltic concrete (AC) and two items surfaced with portland cement concrete (PCC), were considered in this study. Simulated aircraft traffic was applied to the test sections with 200- and 240-kip dual-tandem assemblies. Visual observations of the behavior of the items were recorded throughout the traffic period and were supplemented by photos and rod and level readings. After the completion of traffic, additional tests were conducted by excavating trenches across the traffic lanes.

In addition, data from previous studies, which incorporated stabilized layers and high-quality unbound materials (15 AC-surfaced items and 1 PCC-surfaced item), were used in this study. Single-wheel loadings of 50 and 75 kips, dual-tandem loadings of 160 and 200 kips, and a 12-wheel loading of 360 kips were used in trafficking these items. The general approach and methods of testing were nearly identical for all test sections.

### **9.5.2 FLEXIBLE PAVEMENT ANALYSIS**

Two methods of analysis were applied to the data from the AC-surfaced items: one based on layered elastic theory and the other on comparisons of performance of the various test items. The theoretical analysis method involved the use of computerized models to predict the performance of each item. Response parameters used as input to the models included the resilient modulus as determined from resilient tri-axial tests, the average modulus from unconfined compression tests, and the tensile modulus from indirect tensile tests. The performance predictions were made based on computed subgrade stresses and surface deflections and on limiting subgrade strain criteria. Results of the analysis showed that the stiffness of the stabilized material in the various items was considerably lower than the resilient stiffness of laboratory specimens. It was not possible to develop a transfer between laboratory material characterization and field material properties; thus, a theoretically based design procedure could not be perfected as a part of this study.

The comparative performance method involved a comparison of results for the various items with conventional flexible pavement criteria. The subgrade strength, environmental conditions, and loading conditions were held relatively constant in the items, while the quality of material in the items was varied. A direct comparison of material quality and pavement performance was therefore possible. To account for the differences in performance of the items, equivalency factors were developed which consist of a ratio of the required thickness of a conventional base or subbase layer to the required thickness of high-quality material.

The approach in developing the equivalency factors was to compute the thickness of conventional flexible pavement which would perform in the same manner as the test item with the high-quality layer. The conventional flexible pavement was then assumed to have the minimum requirements of 3 in. of AC and 6 in. of base with the remainder of the structure being subbase. The thickness of the high-quality material was then

ratioed to the thickness of the conventional base or subbase to arrive at an equivalency factor. After equivalency factors were calculated for all test items, values were selected to represent several soil types depending on the type of material and/or stabilizing agent. In each case the resulting high-quality material must meet specified strength and durability requirements before the equivalency factor can be used in design.

#### 9.5.3 RIGID PAVEMENT ANALYSIS

The basic analysis of the PCC-surfaced items also consisted of a comparison of performance with present criteria which involved use of the Westergaard analysis to determine the benefit derived from use of stabilized layers. This analysis did not concern itself with high-quality unbound (nonstabilized) materials since the modulus of foundation reaction  $k$  is used to assess the strength of these type materials in rigid pavement design. This was accomplished by calculating the thickness of PCC pavement required to support the test traffic on actual nonstabilized material. This thickness was then compared to the actual thickness of PCC used in the test items, thereby showing that the stabilized soil layer was equivalent to some thickness of PCC pavement.

Design criteria were developed using the Westergaard and the layered elastic analysis. Using the Westergaard analysis, the stresses at the bottom of the slabs in the test section were computed. These stresses were then used as input to the multilayered elastic analysis, and an equivalent thickness of PCC on a nonstabilized subgrade was computed. Once this relationship had been developed, the layered elastic program was used to extend the data to other slab thicknesses, moduli of elasticity of the stabilized layers, and thicknesses of stabilized layers.

#### 9.6 CONSTRUCTION PROCEDURES FOR STABILIZED LAYERS

The construction of stabilized soil layers requires the use of proper equipment and techniques. As a result of experience gained from

constructing test sections, from field observations, and from a literature study, procedures were developed that present the proper techniques and equipment for mixing, compacting, finishing, and curing of stabilized soil layers.

#### 9.7 SUMMARY OF FINDINGS AND CONCLUSIONS

In the theoretical analysis of the AC-surfaced, high-quality structural layer items, a transfer from laboratory material characterization to field material properties was not accomplished; therefore, the design procedure was developed based on comparative performance with conventional flexible pavements. Experience gained during this portion of the study did show, however, the applicability of using layered elastic theory in establishing correlations between computed pavement response parameters and pavement performance.

The comparative performance analysis of the flexible pavement items resulted in the development of equivalency factors which can be used to compute thickness requirements for pavements having high-quality layers. These equivalency factors together with existing conventional flexible pavement criteria constitute the design criteria for incorporation of high-quality structural layers in flexible airport pavements.

The comparative performance analysis of the PCC-surfaced items also resulted in the development of a thickness design procedure. In this procedure, the required thickness of PCC without an underlying stabilized layer is reduced according to the thickness and quality of the stabilized layer.

Construction techniques were developed for constructing stabilized soil layers.

## CHAPTER 10: MEMBRANE-ENCAPSULATED SOIL LAYERS

### 10.1 APPLICABLE PUBLICATIONS

This chapter summarizes applicable parts of Report No. FAA-RD-73-198, "Comparative Performance of Structural Layers in Pavement Systems," Volume I, "Design, Construction, and Behavior Under Traffic of Pavement Test Sections," June 1974, and Volume III, "Design and Construction of MESL," December 1974.

### 10.2 BACKGROUND

The concept of protecting a soil layer in a pavement system from the intrusion of excess moisture is not new. One application of this concept is a membrane-encapsulated soil layer (MESL). A MESL consists of compacted soil (natural subgrade or locally available soil) placed between lower and upper waterproof membranes that are joined and sealed along the edges. A cross section of a typical MESL pavement section is shown in Figure 10-1.

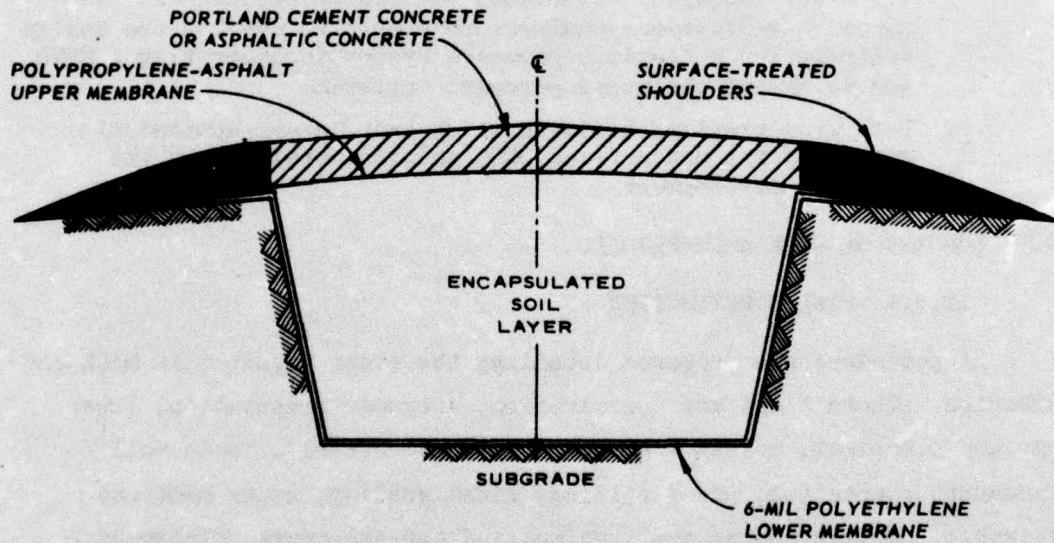


Figure 10-1. Typical cross section of a MESL airport pavement

The major drawback to extensive employment of such pavement systems has been the unavailability of durable, low-cost membrane materials. However, recent studies conducted at WES have shown that satisfactory materials for this purpose are now available at reasonable cost. During this period, valuable experience has also been gained in MESL construction.

#### 10.3 PURPOSE

The purpose of this study was to develop design and construction procedures for the use of MESL's in flexible and rigid airport pavement systems.

#### 10.4 SCOPE

The study consisted of:

- a. Review of previous studies to determine construction techniques and material and equipment requirements for use of MESL's in airport pavements. Based on this review, a procedure for constructing MESL airport pavement systems under normal weather conditions was prepared.
- b. Data from full-scale test sections incorporating MESL test items were analyzed to develop the design procedures. Comparative performance analyses were used to develop the design criteria for a flexible pavement system incorporating a MESL and to extend the rigid pavement criteria.
- c. Data from previous tests in which traffic was applied directly on the nonsurfaced MESL section were used in the criteria development.

#### 10.5 SUMMARY OF WORK ACCOMPLISHED

##### 10.5.1 MESL CONSTRUCTION

A procedure was prepared detailing the steps required in MESL construction. These steps are: excavation, subgrade preparation, lower membrane placement, moisture adjustment (if necessary), loose soil placement, compaction, proof rolling, final grading, upper membrane placement, blotter course application, and surface course placement. Recommendations were also formulated with regard to appropriate soil tests, equipment, and materials to be used in MESL construction.

#### **10.5.2 FLEXIBLE PAVEMENT ANALYSIS**

The performance of two full-scale test items was used to develop a thickness relationship between a pavement system incorporating a MESL and a conventional flexible pavement system. The conventional item consisted of a 3-in. asphaltic concrete surfacing, 6-in. crushed stone base, and 15-in. gravelly sand subbase over a 4-CBR heavy clay subgrade. The MESL item consisted of a 3-in. asphaltic concrete surfacing and a 21-in. MESL constructed from lean clay over a 4-CBR heavy clay subgrade.

Both items were trafficked with a load cart simulating one 12-wheel main gear of a C-5A aircraft. A 360,000-lb loading and a 285-sq-in. tire contact area were used for the load cart.

The conventional item sustained 10<sup>4</sup> coverages of this loading to failure, and the MESL item sustained 1973 coverages to failure. Due to the composition of the two items and the identical loadings, it was possible to directly compare their performances and thereby develop an equivalency factor for relating required thicknesses of MESL and conventional construction. However, because of the limited amount of full-scale aircraft loading test data, it is recommended that the equivalency factor not be applied to MESL design for civil airport pavements. Instead, it was recommended that the MESL be considered as a 1-to-1 replacement with currently specified subbase materials. Thus, the required total thickness of MESL and asphaltic concrete surfacing should be determined from the appropriate flexible pavement design curves. Since unbound coarse-grained material should not be placed on the surface of the upper membrane because of the danger of puncturing the membrane, the thickness of asphaltic concrete surfacings will be equal to the required minimum pavement thickness plus the minimum base course design thickness.

#### **10.5.3 RIGID PAVEMENT ANALYSIS**

The performance of one test item was analyzed to determine the effectiveness of a MESL base course under rigid pavement. This item consisted of 7 in. of fibrous concrete and a 20-in. lean clay MESL over

a 4-CBR heavy clay subgrade. Under both 200- and 240-kip dual-tandem traffic loadings, this item performed better than current criteria would have predicted for an equal thickness of conventional rigid pavement, indicating that the MESL served adequately as a base course. Based on this finding, it was possible to extend the current rigid pavement criteria to establish criteria for MESL construction. The design procedure involves direct testing for the modulus of soil reaction  $k$  on top of the MESL, not correcting for saturation, and comparing this value with the subgrade  $k$  value adjusted to account for an equal thickness of granular base placed between the pavement slab and the subgrade. The lesser of the two  $k$  values is then used with the appropriate rigid pavement design curves to determine the required slab thickness.

#### 10.6 SUMMARY OF FINDINGS AND CONCLUSIONS

Design and construction criteria were developed for use of MESL's in flexible and rigid airport pavement structures. In addition, this study demonstrated that a MESL can be used as an alternative to conventional base and subbase layers in airport pavements. Use of MESL construction is justified based on its potential for lowering construction costs through employment of poorer quality *in situ* foundation materials.

## CHAPTER 11: INSULATING LAYERS

### 11.1 APPLICABLE PUBLICATIONS

This chapter summarizes applicable portions of Report No. FAA-RD-73-198, "Comparative Performance of Structural Layers in Pavement Systems," Volume I, "Design, Construction, and Behavior Under Traffic of Pavement Test Sections," June 1974, and Volume IV (in preparation), "Analysis of Insulating Layers in Pavement Test Sections."

### 11.2 BACKGROUND

A pavement design concept (primarily for highways) used in regions where frost action is a problem is to place insulating layers in the pavement structure to prevent the penetration of freezing temperatures into frost-susceptible materials. Innovations in material utilization have included the use of prefabricated polystyrene panels and polystyrene bead concrete as the insulating layer.

A significant factor in the design of pavements containing insulating layers is the depth at which the insulating layers are placed. To be most effective as insulation, the insulating material should be located as near the surface as possible; the location of the layer should be controlled by the strength of the material and the structural adequacy of the resulting pavement system.

### 11.3 PURPOSE

The purpose of this study was to evaluate the structural performance of prefabricated polystyrene panel and polystyrene bead concrete layers in both rigid and flexible pavement systems, and to develop guidance for locating the layers within the pavement system, based on structural requirements.

### 11.4 SCOPE

The study involved construction and testing of full-scale flexible and rigid pavements, and analysis and evaluation of the response and performance of the pavements when subjected to simulated aircraft loadings. Material tests were performed on the insulating materials

to supplement the pavement tests. Eight flexible pavement test subitems, four containing STYROFOAM\* panels (with compressive strengths of 60 and 120 psi) and four containing STYROPOR\*\* concrete (42pcf), were tested. Eight rigid pavement test subitems, six containing STYROFOAM panels (with compressive strengths of 35 and 120 psi) and two containing STYROPOR concrete (38 pcf), were tested.

#### 11.5 SUMMARY OF WORK ACCOMPLISHED

The flexible pavement test subitems are shown in Figure 11-1. Controlled, accelerated traffic was applied to all eight subitems with a 50-kip single-wheel assembly having a tire inflation pressure of 190 psi. The rigid pavement test subitems are shown in Figure 11-2. Controlled, accelerated traffic was applied to four subitems with a 200-kip load and to four subitems with a 240-kip load. The loads were applied with a dual-tandem assembly having tire inflation pressures of 190 and 250 psi, respectively.

Laboratory tests were conducted on the insulating materials to determine their load deformation and strength characteristics. Unconfined compression tests and repeated load triaxial tests were performed on the STYROFOAM. Results from the unconfined compression tests indicated that the manufacturer's specified compressive strengths of 35, 60, and 120 psi were obtained at approximately 2 percent strain. The repeated load triaxial tests indicated that the resilient modulus of the material was insensitive to the confining and deviator stress and that the 60-psi material had a modulus of about 4500 psi while the 120-psi material had a modulus of about 7000 psi. Penetration tests (using CBR piston) with the 4-in.-thick panels placed on a firm surface also indicated that the stiffness of the material was proportional to the compressive strength. Tests indicated that Poisson's ratio of the material was approximately zero.

Compressive and flexural tests were conducted on the STYROPOR concrete. The tests indicated that the strength and modulus of elasticity

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\* Dow Chemical Company trade name.

\*\* BASF Wyandotte Corporation trade name.

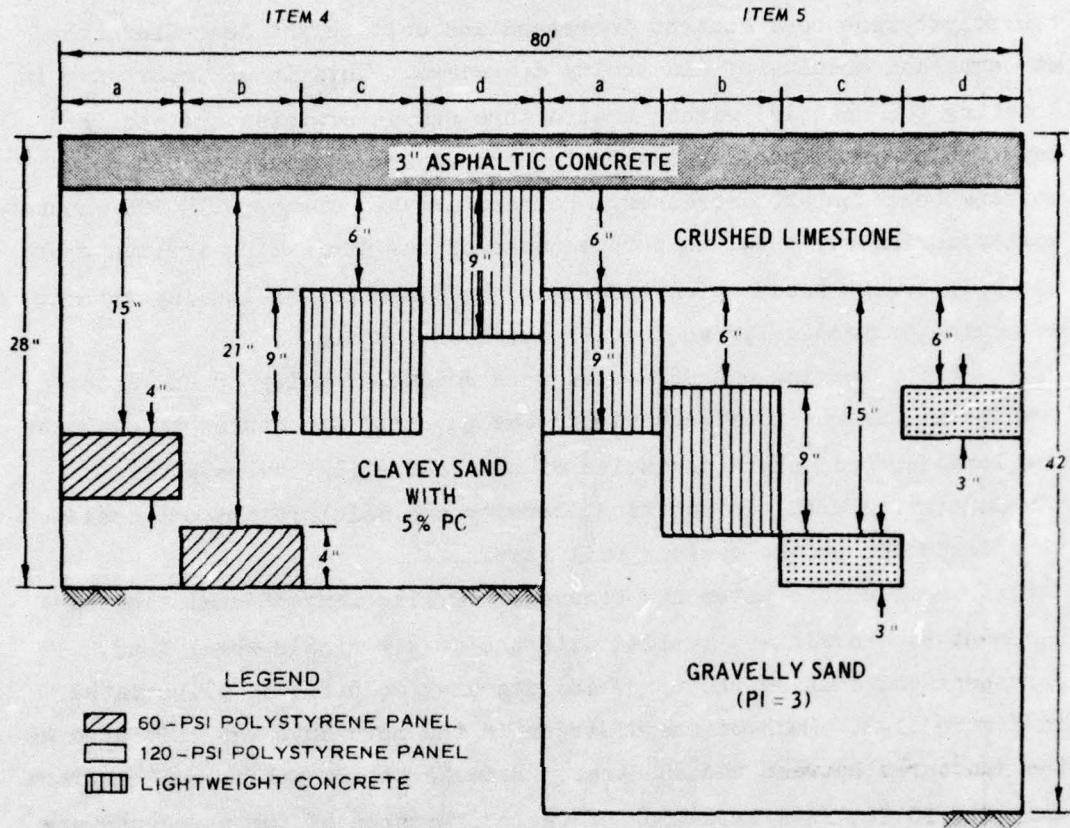


Figure 11-1. Profiles of items in the flexible pavement test section

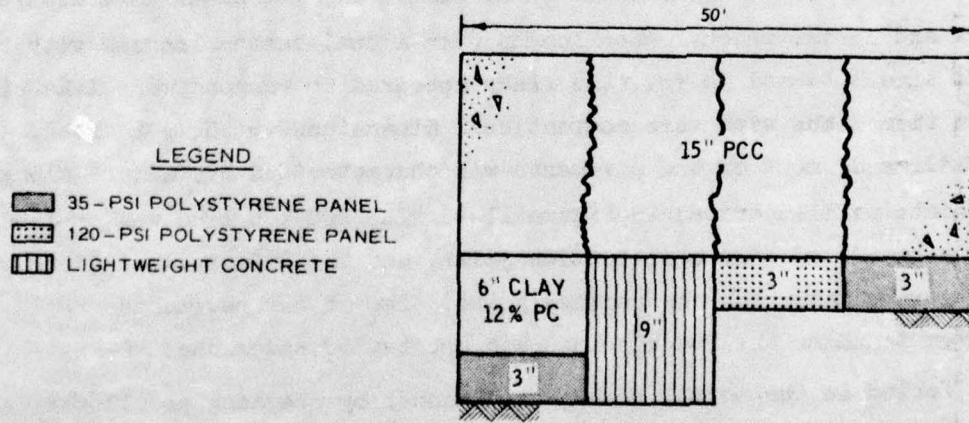


Figure 11-2. Profiles of items in the rigid pavement test section

of the material were proportional to the unit weight of the material. As the polystyrene bead content increased and unit weight decreased, the strength and modulus of elasticity decreased. This is of importance in locating the material within a structure and determining the required layer thickness since the insulating properties decrease as the polystyrene bead content decreases. Poisson's ratio of the STYROPOR concrete was approximately equal to zero because of the dominating influence of the polystyrene beads which deform in the direction of loading without deforming perpendicular to the direction of loading.

Plate bearing and CBR tests were conducted prior to and after traffic at various locations within the pavement structures to evaluate the load-carrying characteristics of the insulating layers within a pavement structure. In addition, density and moisture content tests were conducted on the various soil layers.

The flexible pavements behaved much like conventional flexible pavement as traffic was applied with the 50-kip single-wheel load. Permanent deformation (rutting) and cracking occurred as illustrated in Figure 11-3. Much of the distress in the pavements was initiated at the junctures between the subitems. Because the length of each subitem was only 10 ft, interpretation of the performance of the pavements was difficult.

It is believed that the size of the rigid pavement test sections may also have adversely affected their behavior. The slabs were 12-1/2 ft square and 15 in. thick. When loaded with a dual-tandem loading with the wheels spaced 44 and 58 in., the slabs appeared to respond more like rigid bodies than slabs with more conventional dimensions which would bend. The failure of most of the pavements was characterized by failure along the joints as illustrated in Figure 11-4. The 200-kip load was applied along a keyed and tied construction joint, and the 240-kip load along an aggregate interlock contraction joint. Two of the pavements (subitem 5c under the 200-kip load and subitem 5d under the 240-kip load) failed in the normal or expected manner by cracking as illustrated in Figure 11-5.

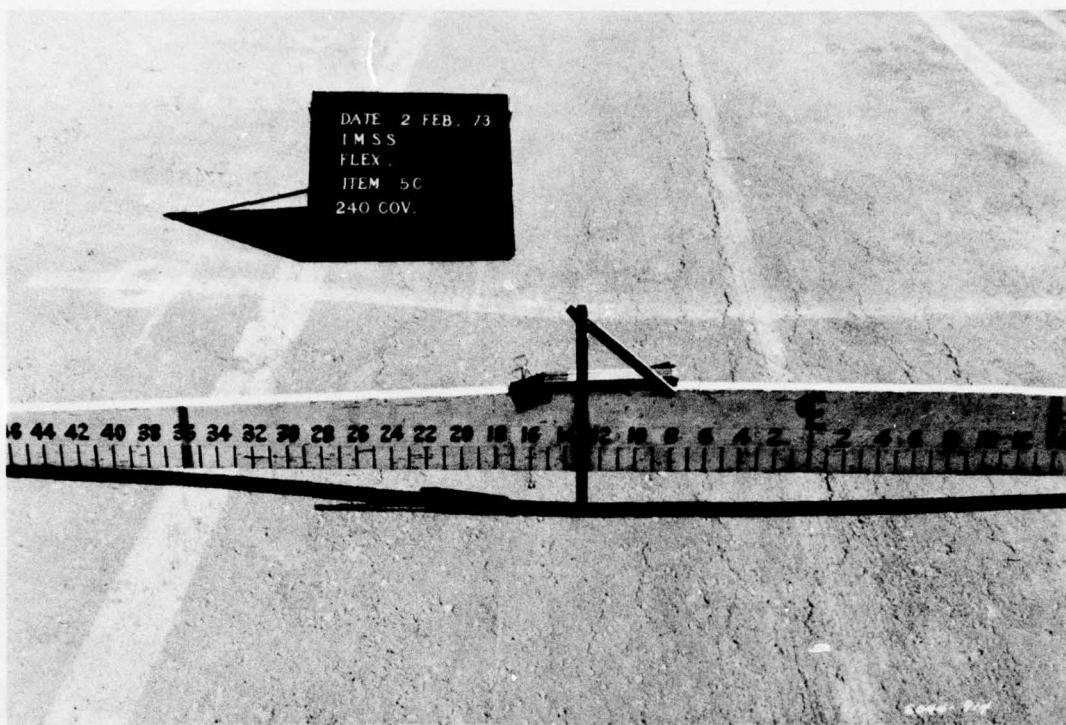


Figure 11-3. Rutting and longitudinal cracking directly above joints between STYROFOAM panels in flexible subitem 5c



Figure 11-4. Typical joint failure along longitudinal construction joint in rigid subitem 5a



Figure 11-5. Normal mode of failure due to cracking rigid subitem 5c

The performance of the flexible pavements was analyzed and compared with existing criteria for conventional flexible pavements. Based on more theoretical considerations, the stresses within the insulating layers were compared to the strength of the material and the load distributing characteristics were considered by analyzing the stresses imposed on the subgrade. A theoretical analysis of the rutting of the pavements was performed, and computed permanent deformations were compared with measured values.

The performance of the rigid pavements was analyzed and compared with existing criteria for conventional rigid pavement. The supporting characteristics of the insulating layers were analyzed by comparing measured plate load-deflection curves with curves computed with the properties of the insulating materials. An analysis of the stresses

and deflections of the concrete slabs was conducted to determine possible causes for the joint failures.

#### 11.6 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Failures in the flexible pavements were initiated at the transitions between subitems. In the subitems containing STYROFOAM layers (4a, 4b, 5c, and 5d), deterioration progressed because of inadequate compaction of the material above the STYROFOAM layers. The STYROPOR concrete provided a stiffer platform against which to compact, and adequate compaction was obtained in the materials above the STYROPOR concrete. Failure in subitems 4c and 4d was attributed to inadequate cover above the STYROPOR concrete which resulted in failure of the STYROPOR concrete. Subitems 5a and 5b contained STYROPOR concrete and performed remarkably well, considering their sizes.

It is recommended that location of insulating layers within a flexible pavement structure be based on limiting the vertical compressive stress on the insulating layer to specified tolerable limits. The limiting magnitude of vertical compressive stress should be less than the measured compressive strength divided by an appropriate safety factor. In addition, published criteria for conventional flexible pavement design should be checked, i.e., vertical strain in the top of the subgrade and horizontal tensile strain in the bottom of the asphaltic concrete surface course.

The rigid pavements did not perform as expected. Joint failures occurred in most of the items and the performance was in general poorer than would have been predicted using measured material properties and current design criteria. All of the pavements experienced pumping. It is hypothesized that the poor performance and joint failures could be attributed to the small slab dimensions relative to the slab thickness, the large, widely distributed loads, and a smaller-than-predicted effective load-carrying capacity of the insulating layers. It is postulated that the discontinuities in the STYROFOAM layers, poor seating of the panels, and rigidity of the individual panels accentuated pumping.

It is recommended that a sand leveling course be used to insure that STYROFOAM panels are firmly seated. A minimum of 6 in. of base (subbase) course should be placed between the slab and the insulating layer (STYROFOAM panels or STYROPOR concrete). It is recommended that determination of required slab thickness be based on a modulus of soil reaction measured on top of the base course.

For subgrades susceptible to pumping (CH, CL, MH, ML, and OL, and SM and SC where the water table is high or drainage poor), a minimum of 4 in. of base course should be placed between the STYROFOAM panels and the subgrade.

## CHAPTER 12: STATISTICAL QUALITY CONTROL PROCEDURES

### 12.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-73-199, "Statistical Quality Control Procedures for Airport Pavement Materials," October 1975.

### 12.2 BACKGROUND

Material variability which results in quality or strengths lower than used in design, whether caused by improper material selection, improper handling and mixing, or improper construction, can nullify the benefits of a rigorous airport pavement design. Therefore, reasonable material specifications and quality control procedures which recognize and consider that material variability will exist are essential in pavement construction to insure that the in-place pavement will perform as intended in design.

In general, current specifications and control procedures do not realistically consider material variability and in many instances are unrealistic in that they do not provide the quality, strength, and uniformity required by the design and in some cases are not enforceable. Examples of these are specifications which use average, minimum, or maximum values without also giving reasonable acceptable limits based upon practical experience and sound engineering judgment.

Statistics represents a tool which, when used with practical experience and sound engineering judgment, can provide more reasonable and more enforceable specifications for materials and construction. Statistics, properly applied, can also result in improved control of material properties and construction procedures. Statistical analyses of data collected can assist the engineer in making better decisions regarding needed changes and in making better assessments of the quality of the product produced. All things considered, statistical procedures can assist in the production of an improved final product and may provide greater confidence that the final product will or will not perform its intended purpose.

### 12.3 PURPOSE

The purpose of this study was to recommend changes to materials specifications and construction control procedures based upon improved statistical procedures.

### 12.4 SCOPE

The study consisted of:

- a. Collection of material and construction control test results from a number of FAA and Corps of Engineers (CE) paving projects.
- b. Performance of a statistical analysis to determine the typical distribution of the various material property and construction control parameters.
- c. Development of recommended standard deviations and maximum allowable deviations from averages for pertinent pavement material and construction parameters for inclusion in applicable specifications based upon the statistical analysis of available data.
- d. Development of a recommended quality control plan, with more emphasis on the use of statistical procedures, to insure that specification requirements are being met.

### 12.5 SUMMARY OF WORK

#### ACCOMPLISHED AND CONCLUSIONS

Results of construction control tests were obtained from 30 airfield/airport construction jobs and analyzed to determine the typical distribution of the data. Criteria used in selection of the job data were that the contractor be reputable and capable, and that the completed job be considered satisfactory. In this regard, recommendations of FAA and CE personnel associated with the construction were used. Data collected included pertinent properties of subgrade, lime-treated subgrade, subbase, base, asphaltic concrete, and portland cement concrete; however, data on all of these materials were not available at every construction site. The data were analyzed to determine the average and standard deviations of each parameter which were, in turn, used to recommend allowable maximum deviations from the average for inclusion in applicable specifications. These recommended specification requirements were determined with two primary considerations: (a) that a

quality pavement be constructed and (b) that the specification requirements be realistic. Based upon the methods used, it was felt that the specification requirements could be recommended with confidence.

Statistical quality control procedures for the various types of construction were developed to include the most pertinent parameters pertaining to the quality of the pavement. In addition, the suggested requirements were presented in a form which permits rapid and efficient evaluation of the parameters.

#### 12.6 RECOMMENDATIONS

It was recommended that the specification requirements and quality control procedures be implemented on a trial basis before incorporation into the standard FAA specifications and that all field control data from these trial jobs be kept for future analysis. The data should be analyzed and used to evaluate the suggested specification requirements and make changes as indicated.

## CHAPTER 13: PAVEMENT RESPONSE TO AIRCRAFT DYNAMIC LOADS

### 13.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-74-39, "Pavement Response to Aircraft Dynamic Loads," which consists of the following:

- a. Volume I, "Instrumentation Systems and Testing Program," June 1975.
- b. Volume II, "Presentation and Analysis of Data," September 1975.
- c. Appendix A to Volume II, "Automatic Data Processing," September 1975.
- d. Appendix B to Volume II, "Data," September 1975.
- e. Volume III, "Compendium," June 1976.

### 13.2 BACKGROUND

Reports of pavement distress associated with current commercial aircraft loads and growing concerns over the possibility of detrimental aircraft dynamic load effects on airport pavements persuaded the FAA to sponsor a Lockheed-California Company study described in Report No. FAA-RD-70-19, "Aircraft Dynamic Wheel Load Effects on Airport Pavements," dated May 1970. The Lockheed study consisted of a literature study, computer analyses to determine aircraft loads and pavement responses, scaled pavement tests, and correlations between experimental and analytical data. In general, the Lockheed study concluded that aircraft dynamic wheel loads have a significant effect on portions of airport pavements. Specifically, the study showed that the primary effects that influence pavement response to dynamic loads are:

- a. The increased magnitudes of aircraft wheel loads resulting from aircraft modes of operation, pavement unevenness, and aircraft structural characteristics during moving ground operations.
- b. The dynamic load phenomena associated with the materials used in the construction of both rigid and flexible pavements.

For a given aircraft and level of pavement unevenness, the loads imposed upon a runway can be accurately defined for various ground

operations. On the other hand, there has been a serious void in information necessary to obtain an accurate description of pavement response to dynamic loads.

### 13.3 PURPOSE

This study was undertaken in an effort to provide experimental pavement response data so that the significance of dynamic loads on airport pavements could be evaluated. Specifically, the basic purpose of the study was to determine the relationship between the responses of typical flexible and rigid runway pavements to static and dynamic loads. The requirements to determine the magnitudes of the dynamic loads, to determine the depths of pavement structures affected by static and dynamic loads, and to investigate the relationship between aircraft ground speeds and aircraft dynamic loads were essential elements of this study.

### 13.4 SCOPE

Two full-scale series of tests using instrumented aircraft and both flexible and rigid instrumented runways were conducted to provide data needed. One series of tests was conducted during the cold period of the year (1972) when the average temperature of the top pavement layer was in the range of 35 to 55° F, while the second series was conducted only on the flexible pavement during the hot period of the year (1974) when the average temperature was in the range of approximately 84 to 116° F. An instrumentation system was installed aboard the aircraft to measure and record three components of force of each of the main gear assemblies of the aircraft. Instrumentation systems were installed at various elevations within the flexible and rigid pavement structures to measure the pavement responses to aircraft loads in the form of relative displacements and pressures. The key element in this experimental approach was the recording of a common time base for both the aircraft load measurements and the pavement response measurements. This control provided a means of correlating the responses of the aircraft and the two pavement structures to

within 1 msec. The locations of the two instrumented pavement test sites were selected so that all possible modes of aircraft ground operation could be investigated during the course of the experimental study.

### 13.5 TEST PROGRAM

An instrumentation package was installed in the pavement structures of runways 04-22 and 13-31 at the National Aviation Facilities Experimental Center (NAFEC) Airport, Atlantic City, N. J., at the two sites indicated in Figure 13-1.

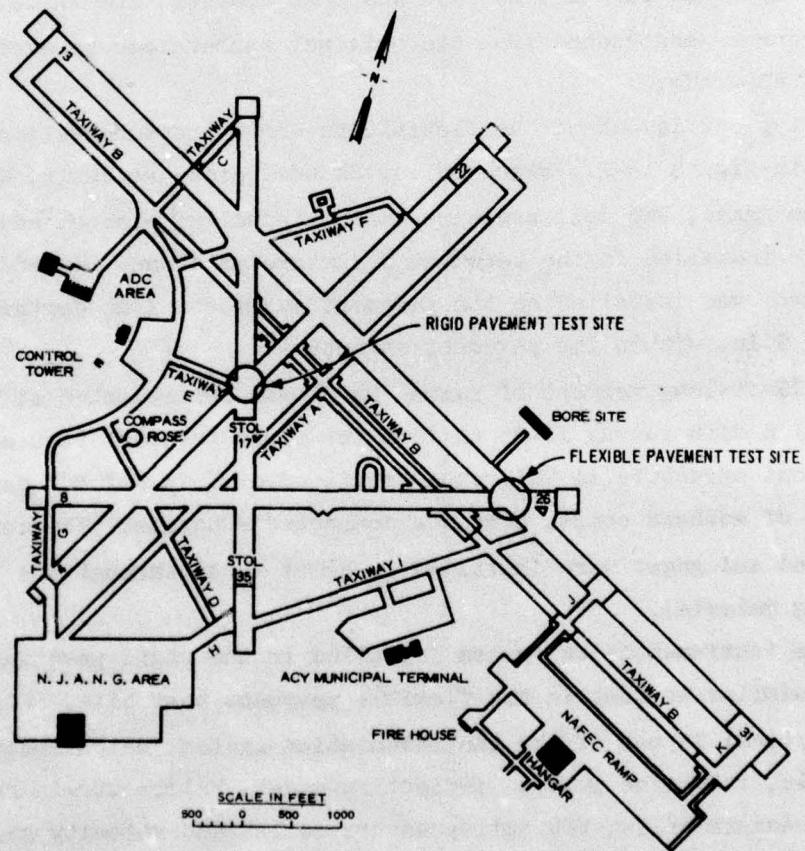


Figure 13-1. Locations of test sites at NAFEC Airport

An 80-ft-long segment of runway 13-31 located at its intersection with runway 8-26 was selected as the flexible pavement test site. This test site was chosen to enable the collection of typical response measurements during landing and at the point of rotation for takeoff as well as during low- and high-speed taxiing, braking, and turning operations. This particular site was in a portion of the runway which was being reconstructed, a factor of great benefit during installation of instrumentation. After reconstruction, the flexible pavement structure in this area consisted of 3 in. of bituminous surface course, 6 in. of bituminous base course, 9 in. of base course consisting of the crushed and mixed original pavement surface and base courses, and 12 in. of subbase course constructed from the original subbase course over a compacted subgrade.

A typical layout of the flexible pavement instrumentation system is shown in Figure 13-2. Bison coils, SE soil pressure cells, WES reflection gages, WES soil pressure cells, inductive probes, and velocity gages were installed in the pavement structure as shown. In addition, a thermistor was installed on the pavement surface and at depths of 3, 6, and 9 in. within the pavement structure.

A 72-ft-long segment of runway 04-22 was instrumented at its intersection with runway 17-35 to form the rigid pavement test site. The pavement structure in this area consisted of 7 in. of PCC pavement and 8 in. of subbase course over the compacted subgrade. The concrete was removed and gages were installed in holes cored through the underlying material.

The instrumentation system installed in the rigid pavement test site was similar to that in the flexible pavement test site. Figure 13-3 shows a typical layout of the instrumentation system, which included Bison coils, inductive probes, deflection gages, Valore strain gages, SE soil pressure cells, WES soil pressure cells, and velocity gages. Thermistors were installed on the surface of, at the bottom of, and at a depth of 3.5 in. within two slabs of the rigid pavement test site.

A system of laser light beam sources and detectors was installed along the edges of the runways such that a light beam was projected

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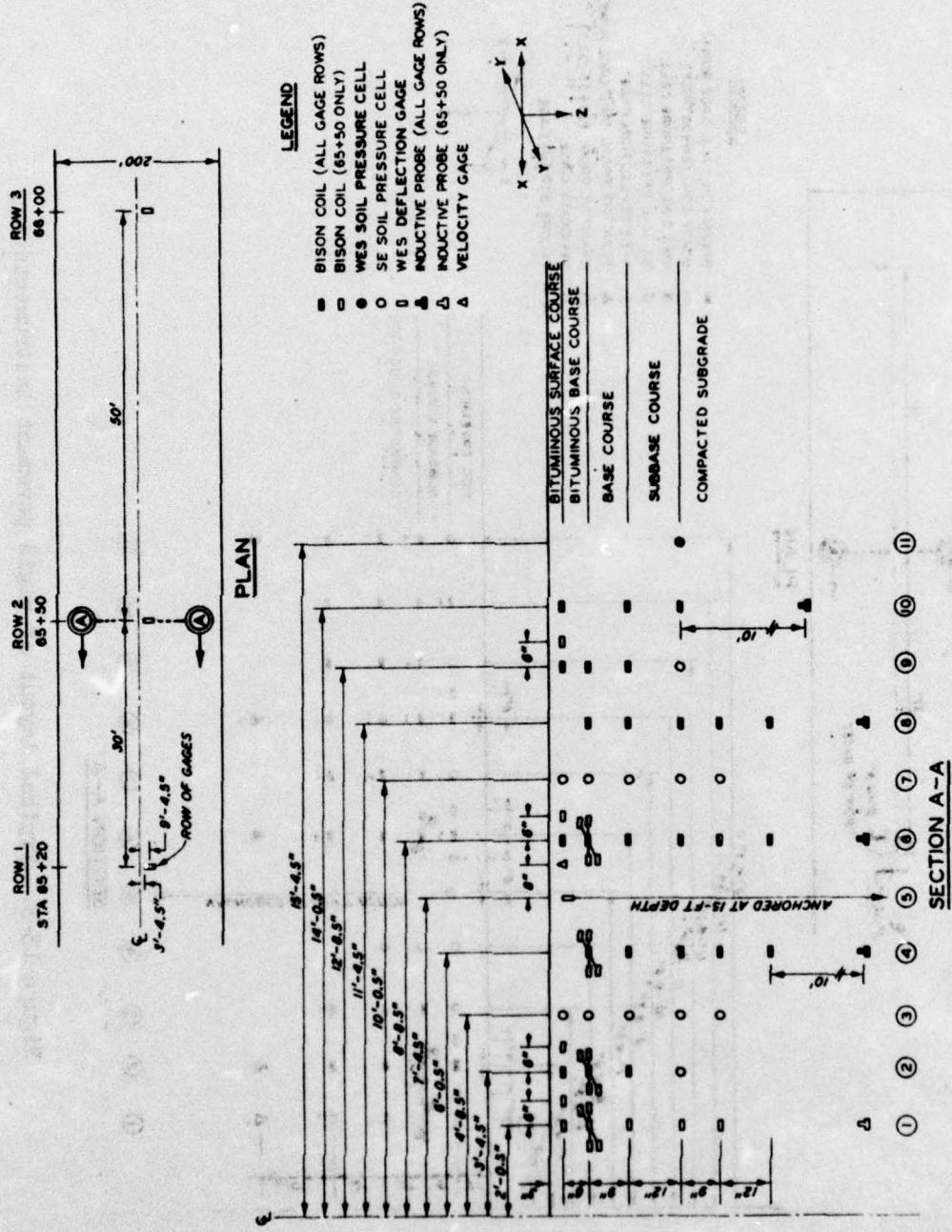


Figure 13-2. Typical layout of flexible pavement instrumentation

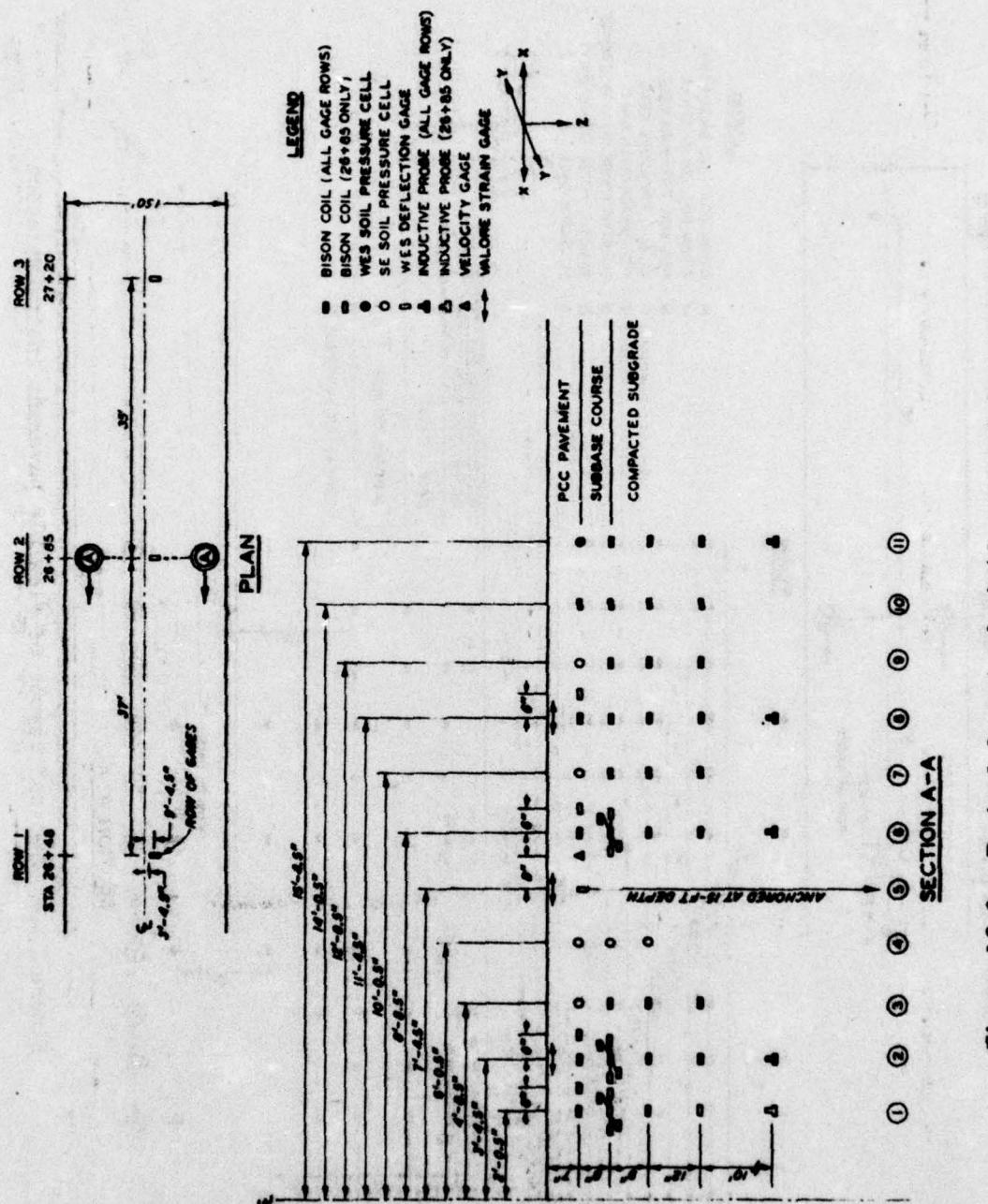


Figure 13-3. Typical layout of rigid pavement instrumentation

directly above and parallel to each instrumentation gage row. An electrical impulse was generated when the wheels of the instrumented aircraft passed between the source and detector, thereby signaling the instant at which the wheels were directly over the gage row. The lateral position of the aircraft was determined by visual inspection of a stripe of flour and water solution painted on the surface of the runways adjacent and parallel to each gage row.

A synchronized common time signal was recorded on both aircraft and ground data tapes. This provided the means by which the pavement response could be correlated with the corresponding aircraft load. With the exception of the thermistors, all instruments were recorded simultaneously on magnetic tapes, and all ground data tapes contained the time code and laser signals. Temperatures were recorded on paper tape.

An instrumented B-727 was leased from United Airlines, Inc., for the cold weather testing of 1972, and a similar B-727 and a C-880 were obtained from the FAA and instrumented by WES for the warm weather testing of 1974. The aircraft were instrumented to measure the force transmitted to the pavement structure. On-board instrumentation for all three aircraft included signal conditioning equipment, a time code generator (synchronized with the ground time code generator for correlation of test results), and a 14-track analog magnetic tape recorder.

Data were collected for 408 aircraft operations during the cold weather tests. Of this total, 203 operations were on the flexible pavement test site and the remaining 205 were on the rigid pavement test site. During the warm weather tests, data were collected for 281 aircraft operations on the flexible pavement test site.

The following types of tests were performed during both cold and warm weather tests:

- a. Static load tests. The aircraft was positioned over each gage row and data collected. These tests provided data for comparison with data from dynamic load tests as well as a check of the capability of the instrumentation system.
- b. Dynamic load tests. Various aircraft ground operations were conducted on the test sites and data collected. Pavement responses and aircraft dynamic loads were determined under the following aircraft operating modes:

- (1) Creep-speed taxi (3 to 8 knots).
- (2) Low-speed taxi (15 to 30 knots).
- (3) Medium-speed taxi (45 to 80 knots).
- (4) High-speed taxi (85 to 130 knots).
- (5) High-speed braking (130 to 45 knots).
- (6) Takeoff rotation (85 to 130 knots).
- (7) Touchdown.
- (8) High-speed braking with reverse thrust.
- (9) Turning (4 to 30 knots).

### 13.6 ANALYSIS OF DATA AND RESULTS

In general, all instrumentation performed satisfactorily. The instrumentation responses were reduced by automatic data processing (digital) techniques. Interpretation of the processed data revealed the following:

- a. Elastic (including viscoelastic) and inelastic phases of material behavior were found acting in the displacements of both flexible and rigid nonconditioned pavements. (Conditioning is a test procedure in which a pavement point is loaded repeatedly before recording instrument responses. By conditioning, inelastic response, which can be erratic, is made to approach zero; therefore, instrument response then appears to be "stable." While this type conditioning temporarily eliminates the inelastic movements, it is not really representative of behavior under actual traffic loading, since traffic is randomly distributed and approaches a normal distribution with time.) The magnitude and direction of movement of the inelastic response were controlled by the gear-to-gage offset distance. Changes in direction of the inelastic response and the upward movement at the various offsets and in the immediate gage vicinity occurred. However, the elastic and inelastic responses exhibited symmetry and repetition. Inelastic response was symmetric and repetitive for a given symmetric or orderly load sequence.
- b. In order to be able to fully interpret and analyze the nonconditioned pavement structure response data, the elastic and inelastic phases had to be separated (they occur simultaneously) and treated independently in the investigation of static and dynamic load test results. Instrument responses could not be completely analyzed unless the inelastic behavior was fully recognized and utilized.

- c. Two different types of displacement responses were identified as acting in both flexible and rigid pavements. The two types are total pavement structure response as assumed to be referenced to infinity (inertial reference) and individual pavement structure element response referenced internally to each element (noninertial reference). Each type of response exhibited both elastic and inelastic material behavioral phases.
- d. Bow waves in front of the wheels and elastic vertical expansions behind and adjacent to the wheels were found to occur within the structural elements (noninertial reference) of both pavement structures under moving aircraft operations.
- e. The vertical pressure data for both flexible and rigid pavements were found to be totally recovered, elastic (corresponding to the elastic phase of behavior), upon removal or passage of a load. No residual pressures appeared to be acting; therefore, the inelastic behavior did not seem to induce residual vertical pressures. The pressure cells appeared to be carried with or ride within the pulsating structures.

Pavement response to aircraft dynamic loads resulted in the following findings:

- a. B-727 aircraft dynamic load tests in 1972 (cold weather) and 1974 (warm weather) on the nonconditioned flexible pavement structure and in 1972 on the nonconditioned rigid pavement structure at NAFEC showed that no basic aircraft ground operating mode induced pavement responses (elastic plus inelastic) greater than those occurring for static load conditions even though the aircraft dynamic loads were as large as 1.2 times the static load. Elastic response alone generally indicates this also to be true. The pavement surfaces were relatively smooth in the test site areas.
- b. However, extrapolation of the test results indicates that for stiff pavement structures, such as the rigid pavement and the flexible pavement in cold weather, unusual conditions of large dynamic loading that could result from rougher surfaces than at NAFEC, holes or bumps, etc., could possibly cause responses larger than those that would occur under static loading. This behavior is possible because of the inelastic behavior being of low magnitude for the stiff pavements and the elastic response being essentially of a constant magnitude with rate of load applications. The larger than static load response that could occur should be entirely elastic and should not be detrimental to the pavement structure except by contributing to an increase in elastic fatigue damage.

- c. Based upon gradually reduced elastic response but primarily upon reduced inelastic response with high speeds, indications are that thickness can be reduced in the interior of runways.
- d. Measured aircraft loads during turns showed that high horizontal loads are applied to the pavement surfaces. Due to the high loads and to prevent excessive deterioration in turn areas, the pavement in exit areas of flexible pavement runways should be strengthened or be stronger than the main runway.
- e. Test results showed inelastic behavior to be highly dependent on temperature, rate of load application, and load history (magnitude of load and lateral position of aircraft).
- f. Inelastic displacements larger than the elastic displacements were measured within the velocity range of static load to low-speed taxi.
- g. Test results showed elastic behavior to be almost constant for stiff pavement structures (rigid and low-temperature bituminous pavements) and the probable viscoelastic effects to be more pronounced at high temperatures in bituminous materials.
- h. The flexible pavement structure layer at a depth of 39 to 51 in. slightly responded (less than 10 percent of the surface response) to the various modes of aircraft operation. The rigid pavement structure layer at a depth of 15 to 24 in. responded (about 30 percent of the surface response) to the various modes of aircraft operation. These were the deepest layers monitored during dynamic load tests for both pavement structures.
- i. Elastic and inelastic displacement behavior and response can be accurately mathematically modeled.
- j. The elastic and inelastic displacement behavioral phases directly associate the behavior of WES pavement test sections under simulated aircraft loads and wheel configurations and distributed (not conditioning) traffic to actual pavement behavior under actual aircraft operations (NAFEC tests). This connection means that any further investigation of dynamic load effects can probably be conducted on pavement structure test sections of limited size.
- k. Inelastic behavior occurred in both the nonconditioned flexible and rigid pavement structures and may possibly be a common characteristic that links or ties together the performance of all pavement types. In fact, it may be the major controlling factor or mechanism for pavement performance and life because it can be the primary movement for static and low-speed operations.

### 13.7 RECOMMENDATIONS

Based on the study of pavement responses to aircraft dynamic loads, the following recommendations are made:

- a. The required thickness of pavements subjected to parked or slow-moving aircraft should be based upon the static weight of the aircraft, as is the current practice. This is considered to include the parking aprons, taxiways other than high-speed exit areas, and runway ends. In high-speed exit areas, runway interiors, and other areas that are subject entirely to high-speed aircraft operations, the design should be based upon an analysis of the dynamic loading to the pavement and upon the pavement response to dynamic loading. In high-speed exit areas, high horizontal loads are applied to the pavement surface and should be considered in pavement design. Due to the large loads and thus the likelihood of excessive deterioration in turn areas, the pavement surface in exit areas of flexible pavement runways should be strengthened or be stronger than the main runway. In runway interiors, the NAFEC test data indicate that thickness reductions can be considered. In order to take full advantage of the NAFEC test data in pavement design, more knowledge is needed concerning pavement failure mechanisms and deterioration growth functions and causes.
- b. A vast amount of good data were collected, but an analysis of the data beyond the objectives of this report has not been made. There is a wealth of information to be gained, and the analysis and study of the data and results should be continued. Specific areas of study should be the elastic and inelastic displacement phases with emphasis on further defining and understanding the inelastic behavior and its importance to pavement structure performance.
- c. Development of a mathematical model or models of the elastic and inelastic behavioral phases should continue with emphasis on correlating and defining the functions of the constant coefficients for the model presented. These coefficients should be functions of variables such as depth, load, number of wheels, structure strength, temperature, rate of load application, etc.
- d. Constitutive relations based on the measured results for the pavement structures should be investigated.
- e. Results pertaining to longitudinal moving wheel displacement basin responses should be investigated with emphasis on developing a mathematical model or models, based on the measured results, for purposes of simulating rolling aircraft wheels on a pavement.

If satisfactory results concerning the above items are obtained, a solid foundation should exist upon which to base theoretical models concerning pavement structure design, behavior, and performance under any type of loading conditions.

## CHAPTER 14: LONGITUDINAL CONSTRUCTION JOINTS IN RIGID PAVEMENT

### 14.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-72-106, "Strengthening of Keyed Longitudinal Construction Joints in Rigid Pavements," August 1972.

### 14.2 BACKGROUND

The multiple-wheel gears of present commercial aircraft impose loadings on pavements that are significantly different from those previously encountered. Tests of full-scale rigid pavement test sections at WES indicated that thickness requirements, as determined from extrapolations to current design criteria, are adequate for these loadings; however, results of these same tests also demonstrated that conventional keyed longitudinal construction joints are inadequate for weak or soft foundations. Since premature failures of such joints could result in early structural failures of the pavement slabs, developing a method for improving the performance of keyed longitudinal construction joints is of critical importance to the civil aviation community.

### 14.3 PURPOSES

The purposes of this study were to:

- a. Determine the adequacy of conventional keyed longitudinal joints in rigid pavements constructed over medium- and high-strength subgrades when subjected to current heavy wheel loads.
- b. Develop a method of strengthening these joints in existing rigid pavements.
- c. Study the use of doweled longitudinal construction joints in rigid pavements.
- d. Determine the effectiveness of a stabilized base course in providing added protection to longitudinal joints in rigid pavements.

#### 14.4 SCOPE

This study consisted of testing and analyzing the results from four items of a rigid pavement test section representing a range of jointing conditions.

#### 14.5 SUMMARY OF WORK ACCOMPLISHED

The plan and section of the rigid pavement test section are shown in Figure 14-1. Traffic was applied to the test section using a 360,000-lb, 12-wheel assembly representing one main gear of the C-5A aircraft and a 166,000-lb, dual-tandem assembly representing one-half of the B-747 aircraft main gear. Untied keyed and doweled construction joints designed according to Corps of Engineers criteria were used to join the two paving lanes. These joints were essentially the same as the type C-keyed and D-doweled construction joints formerly recommended under FAA design criteria. A brief description of each item and its performance under traffic is presented below.

##### 14.5.1 ITEM 1

This item consisted of an 8-in. plain concrete surfacing constructed on a base composed of 24 in. of pit-run clayey gravelly sand which exhibited an effective modulus of soil reaction  $k$  of 200 pci. The subgrade was a low-strength clay. A keyed joint was used for one-half (25 ft) of the item, and a doweled joint was used for the other half. The keyed joint failed early in the traffic life, but the doweled joint performed satisfactorily for the full life of the item. Indications were that the performance of keyed joints in rigid pavement on medium-strength foundations would be marginal.

##### 14.5.2 ITEM 2

This item consisted of an 11-in. plain concrete surfacing constructed directly on the low-strength ( $k = 75$  to 100 pci) subgrade. A keyed joint was used for the full length of the item (50 ft). Prior to traffic, the keyed joint was strengthened by the methods shown in Figure 14-1. Each strengthening method studied protected the joint

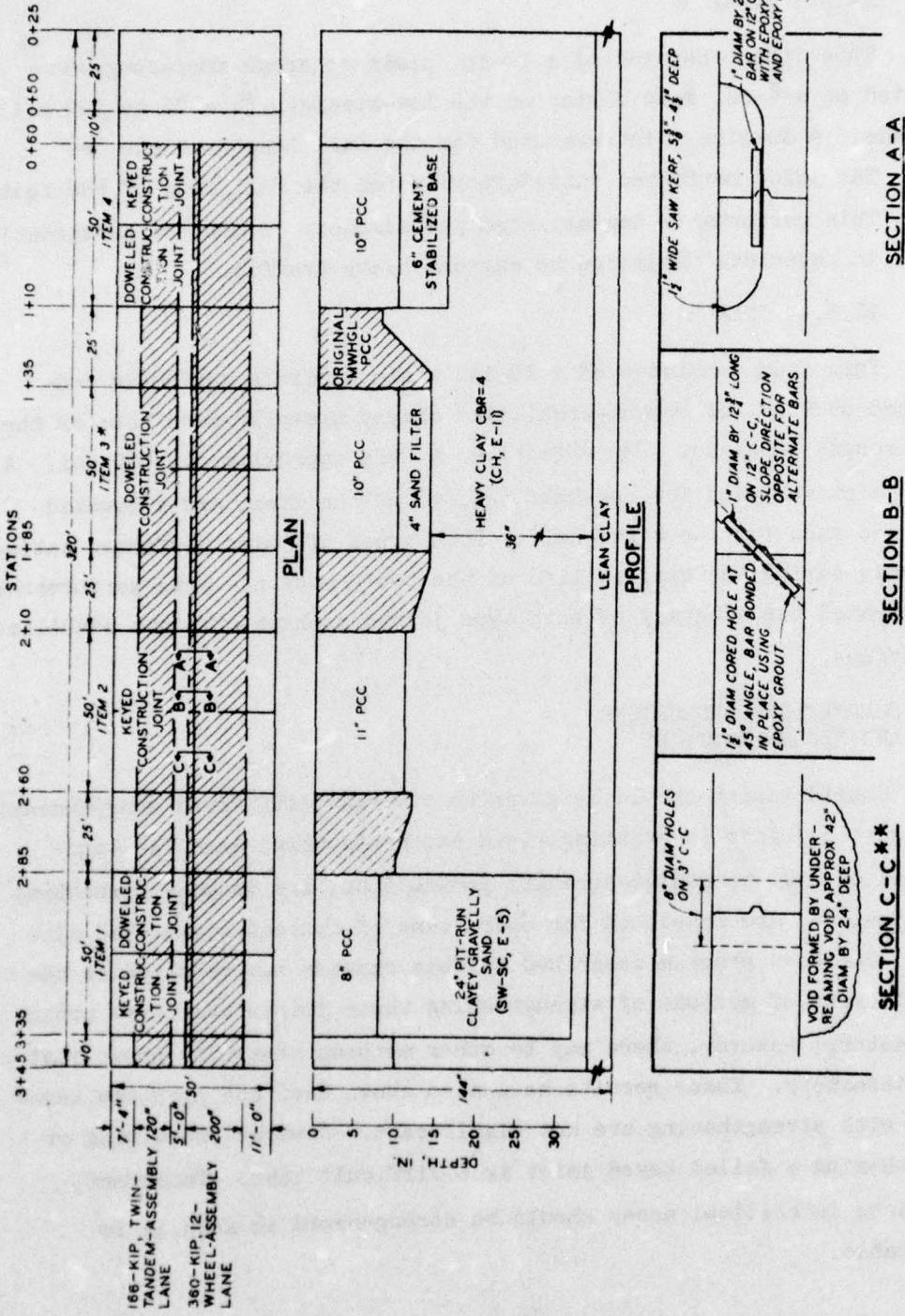


Figure 14-1. Layout of keyed longitudinal construction joint test section

for the full life of the test section. This performance demonstrated the feasibility of strengthening keyed joints in existing pavements that are to be used for current heavy aircraft.

#### 14.5.3 ITEM 3

This item consisted of a 10-in. plain concrete surfacing constructed on a 4-in. sand filter on the low-strength ( $k = 75$  to 100 pci) subgrade. A doweled joint was used for the full length of the test item. The joint performed satisfactorily for the full life of the test item. This performance demonstrated the adequacy of doweled construction joints in pavements subjected to current heavy traffic.

#### 14.5.4 ITEM 4

This item consisted of a 10-in. plain concrete surfacing constructed on 6 in. of cement-stabilized clayey gravelly sand base on the low-strength subgrade. The effective  $k$  was approximately 400 pci. A keyed joint was used for one-half (25 ft) of the item, and a doweled joint was used for the other half. Both types of joint performed satisfactorily during the traffic life of the test section. This performance demonstrated the adequacy of both type joints on high-strength stabilized foundations.

#### 14.6 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Consideration should be given to strengthening keyed longitudinal construction joints in existing rigid pavements which have been constructed on low- to medium-strength foundations, are in good structural condition, and are scheduled for operations of current commercial aircraft. The test program described in this chapter has resulted in the establishment of methods of strengthening these joints that have proven satisfactory; however, there may be other methods that will prove just as satisfactory. These results have also shown that the problems associated with strengthening are not significant. However, repairing or strengthening a failed keyed joint is a difficult task. Therefore, the joints in critical areas should be strengthened as soon as is practicable.

Keyed longitudinal construction joints in existing rigid pavements constructed on high-strength or stabilized soil foundations will probably perform satisfactorily under present commercial aircraft loads. However, it is recommended that these types of pavements be periodically observed (or tested) and, if distress is noted, that consideration be given to strengthening the keyed joints.

For new construction, the use of type C-keyed longitudinal construction joints on unbound foundations is discouraged except in the very low-usage areas, such as the outer lanes of runways and aprons. As an alternate, thickened-edge keyed, thickened-edge butt, or type E-hinged construction joints may be satisfactory. When using the type E-hinged joint, the designer should be aware that tying the construction joints of several successive lanes together may result in longitudinal cracking in the paving lanes since the tie bars have the effect of immobilizing the joint where contraction and expansion would ordinarily occur.

The type D-doweled longitudinal construction joint proved to be satisfactory in the test program described herein and is recommended for use. These results substantiate the good performance of this type joint.

The results of this study have been incorporated in Chapter 3, Section 5, Paragraph 50b, of Advisory Circular AC 150/5320-6B.

## CHAPTER 15: AIRCRAFT-PAVEMENT COMPATIBILITY

### 15.1 APPLICABLE PUBLICATIONS

The following is a summary of "Aircraft-Pavement Compatibility Study," Report No. FAA-RD-73-206, September 1974.

### 15.2 BACKGROUND

Since 1958, the FAA has followed a policy of limiting pavement design for large commercial aircraft to an equivalent 350,000-lb gross weight on a dual-tandem gear configuration. To remain within acceptable stress limitations, the B-747 has been designed with 4 main gear bogies with 16 wheels and the DC-10-10 and the L-1011 have been designed with larger wheels at greater spacings to remain within the flotation criterion. The penalty costs associated with conformance to the design criterion have been hypothesized, but quantification has not previously been made public.

Aircraft gross weights have already begun to exceed the 0.5-million-lb class, and the penalties resulting from the flotation criterion are becoming all too obvious. For instance, the DC-10-20 and the DC-10-30 have two additional wheels under the fuselage. The wide spacing required on the four main gears of the B-747 places the gears beneath the engines, thereby decreasing the torque available for ground turning and greatly impeding the ground maneuverability. As the aircraft industry moves toward aircraft in the 1.5- to 2.0-million-lb gross weight class, even greater penalties would seem probable.

### 15.3 PURPOSE

The purpose of this study was to perform an economic analysis relating pavement upgrading costs to the penalty costs associated with adding gears and wheels to aircraft in order to provide adequate flotation for the current design criterion of 350,000 lb on a dual-tandem configuration.

#### 15.4 SCOPE

The basic approach used in this study was purely economic; environmental, sociopolitical, and energy conservation factors were not considered. A three-element program was undertaken to accomplish the purpose of the study.

First, a contract was let to Lockheed-California Company to design landing gears for two categories of aircraft. Category I consisted of a representative of the relatively new series of commercial jet aircraft (in Lockheed's case, the L-1011). Category II consisted of a projected 1.5- to 2.0-million-lb aircraft. For each of these types of aircraft, Lockheed designed three representative landing gears. The first type gear was constrained by the criterion of causing no more distress to the pavement than a 350,000-lb aircraft with a dual-tandem gear with intended spacings similar to a DC-8-63F aircraft. The second type of gear was one that was optimized with respect to the aircraft without pavement constraints. The third type of gear was a compromise or median gear, causing pavement distress somewhere between the other two gear types. In addition, Lockheed was required to project the major hub airports that would be servicing the two categories of aircraft in the year 1985 and, from derived city pairs, to develop the economic penalties associated with the three gear types for both categories of aircraft.

Based on the gear configurations and parameters provided by Lockheed, WES analyzed the airport master plans for the projected major hub airports and decided whether new construction or overlays were required to accommodate the six combinations of aircraft. Strengthening when required was designed for each major hub airport and costs computed based upon total pavement areas affected. Cost data were obtained from FAA Regional Offices in the form of bid tabulations and associated cross-sectional designs.

The final phase of the study consisted of performing a cost analysis at each major hub airport with respect to equivalent annual cost.

## 15.5 SUMMARY OF WORK ACCOMPLISHED

### 15.5.1 AIRCRAFT COST DEVELOPMENT

Gear types during this portion of the study were optimized with respect to cost instead of weight. Figures 15-1 and 15-2 show the gear designs for the Category I and Category II aircraft, respectively.

The aircraft costs associated with carrying the landing gear weight and volume in excess of that of the optimized gear arise from four sources: acquisition cost, maintenance cost, flight cost, and lost revenue (payload) cost. The first three costs were considered in the landing gear design since the design was based on the least cost design. The lost revenue cost was based upon the lost payload of the aircraft. Figure 15-3 is a graphic illustration of the probability assumptions used for determining lost payload.

Basically, the assumptions include an average weekly payload  $\bar{X}$ , a normal distribution of payload weight about  $\bar{X}$ , and a coefficient of variation of 60 percent. The equations used in the lost revenue model were

$$\text{(total revenue, \$)} = (\text{passenger miles}) \times (\text{yield/ passenger mile}) + (\text{cargo ton miles}) \times (\text{yield/ton mile}) \quad (15-1)$$

$$\text{(total weight, lb)} = [(\text{passenger miles} \times 200 \text{ lb/passenger}) + (\text{cargo ton miles} \times 2000 \text{ lb/ton})] \div (\text{flight distance}) \quad (15-2)$$

$$\text{(average yield, \$/lb)} = \frac{\text{(total revenue)}}{\text{(total weight)}} \quad (15-3)$$

multiplied by 52 weeks per year to arrive at an annual expected lost revenue by aircraft type by distance-block under various landing gear/ operational empty weight assumptions. This lost revenue was then summed over all the distance-blocks analyzed for the projected 26 major hub airports to determine the total annual lost revenue from operations out of major domestic hub airports.

The total point estimate annual costs relative to the optimal gear configurations are shown below.

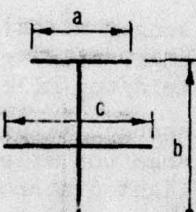
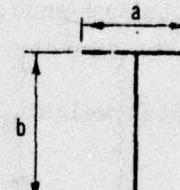
ITEM	CURRENT-PAVEMENT GEAR	MEDIAN-PAVEMENT GEAR	OPTIMIZED GEAR
GEAR CONFIGURATION	6-WHEEL BOGIE	4-WHEEL BOGIE	4-WHEEL BOGIE
TIRE VERTICAL LOAD, POUNDS	38,630	57,950	57,950
TIRE PRESSURE, PSI	200	200	215
TIRE DIAMETER, INCHES	44.8	56.1	53.8
BOGIE SIZE, INCHES	a b c	42.3 97.7 56.4	44.5 59.9 -
BOGIE CONFIGURATION			

Figure 15-1. Gear designs for Category I aircraft

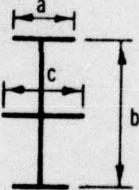
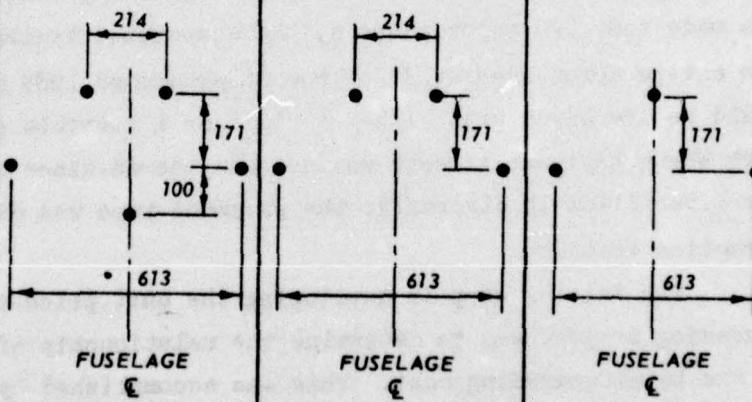
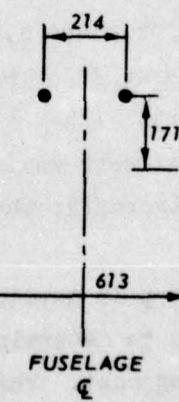
ITEM	CURRENT-PAVEMENT GEAR	MEDIAN-PAVEMENT GEAR	OPTIMIZED GEAR
GEAR CONFIGURATION	FIVE 6-WHEEL BOGIES	FOUR 6-WHEEL BOGIES	THREE 6-WHEEL BOGIES
TIRE VERTICAL LOAD, POUNDS	47,500	59,375	79,167
TIRE PRESSURE, PSI	150	200	250
TIRE DIAMETER, INCHES	56.2	56.9	58.4
BOGIE SIZE, INCHES	a b c	52.2 120.5 69.6	54.1 124.9 72.1
GEAR LOCATIONS, INCHES			
			

Figure 15-2. Gear designs for Category II aircraft

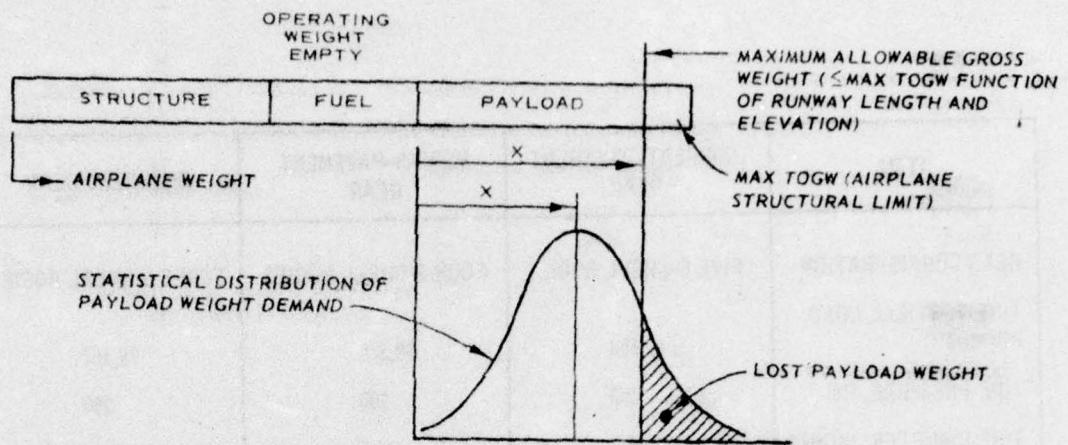


Figure 15-3. Probability assumptions for determining lost payload

	<u>Current Pavement Gear</u>	<u>Median Pavement Gear</u>
Category I aircraft	\$ 6,673,397	\$ 1,929,880
Category II aircraft	<u>68,777,864</u>	<u>35,160,820</u>
Total annual aircraft cost	\$75,451,261	\$37,090,700

#### 15.5.2 PAVEMENT COST DEVELOPMENT

Because of spatial and temporal variables, a statistical approach was used to develop the total pavement upgrading costs. An assumption was made that two major runways, the associated taxiway systems, and the entire apron area at 25 of the 26 projected 1985 major hub airports would be overlayed with either a rigid or a flexible pavement (Dallas-Fort Worth Regional Airport was not considered since it has been designed for 1.5-million-lb aircraft); the pavement type was determined from construction records.

The initial step in developing the unit price for each pavement upgrading project was to determine the relationship of the pavement cost to the total upgrading cost. This was accomplished by analyzing bid tabulations for 14 major airport paving projects published during 1971-1972. The mean  $\bar{X}$  and the standard deviation  $\sigma$  of seven categories as a percentage of the total upgrading cost are shown below:

<u>Category</u>	<u>X</u>	<u><math>\sigma</math></u>
Excavation	13.10	11.08
Pavement	72.79	9.81
Subsurface structures	7.13	5.70
Wiring	1.74	2.27
Lighting	2.21	4.47
Painting	0.37	0.67
Miscellaneous	2.66	4.92

The average price of pavement as a percentage of the total contract price was found to be 72.79 percent, with a coefficient of variation of 14 percent.

A difference between the ratio of rigid pavement price and the ratio of flexible pavement price to total contract price was indicated and a grouped analysis determined the ratios of pavement price to total price to be:

<u>Pavement Type</u>	<u>X</u>	<u><math>\sigma</math></u>
Rigid	77.51	8.03
Flexible	68.06	9.60

The pavement unit prices were developed, insofar as possible, on the basis of the price per square yard per inch. Prices were assumed to vary hyperbolically with thickness within an acceptable range.

The equation used for determining the unit price for pavement was:

$$C = \text{price per square yard} : \text{thickness} \quad (15-4)$$

or, for flexible pavements, when bid tabulations were listed in price per ton,

$$C = \text{cost per ton} \times \frac{1}{2000 \text{ lb/ton}} \times 150 \text{pcf} \times 9 \text{ sq ft/sq yd}$$

(15-5)

$$\times \frac{1}{12 \text{ in./ft}}$$

A list of national average prices for pavement products is given below.

<u>Pavement Product</u>	<u>Cost Unit</u>	<u>Number of Observations</u>	<u>Mean Price <math>\bar{X}</math></u>	<u>Standard Deviation <math>\sigma</math></u>
Portland cement concrete (Item P-501)	\$/sq yd/in.	46	0.94	0.30
Bituminous surface course (Item P-401)	\$/sq yd/in.	21	0.54	0.10
Crushed aggregate base (Item P-209)	\$/sq yd/in.	8	0.19	0.03
Bituminous base (Item P-201)	\$/sq yd/in.	13	0.59	0.22
Prime coat (Item P-602)	\$/sq yd	9	0.07	0.02
Tack coat (Item P-603)	\$/sq yd	23	0.03	0.02

The third step in developing the pavement cost was to design the pavement cross section required for the Category I and II aircraft. FAA design criteria were used for the design at a standard 100,000 aircraft pass level. Only two types of overlays were considered: full-depth bituminous overlays (Item P-401) and PCC overlays (Item P-501). A total expected area of 29,939,536 sq yd was calculated with 32.2 percent consisting of runway area, 23.4 percent consisting of taxiway, and 44.4 percent consisting of apron area.

A comparison of the total aircraft cost and the total pavement upgrading cost was made in terms of equivalent annual cost in 1985 dollars. The basic equation for determining the equivalent annual pavement cost in 1985 dollars can be expressed simply as

$$x = p \times A \times (1 + i)^n \left[ \frac{i(1 + i)^m}{(1 + i)^m - 1} \right] \quad (15-6)$$

where

$x$  = equivalent annual cost of pavement upgrading, 1985 dollars

$p$  = average total cost of upgrading per square yard, dollars

$A$  = pavement area to be upgraded, sq yd

$i$  = interest rate, percent

$n$  = number of years to construction (or bond issuance)

$m$  = amortization period of the pavement structure, years

Some basic value assumptions were necessary in order to make comparisons using this 5-space function. Expected values for  $p$  of \$7.36, \$7.77, \$7.45, and \$12.82 were computed for the Category I median and optimal gears and the Category II median and optimal gears, respectively. The computed value for  $A$  was 29,939,536 sq yd. Assumptions for the remaining independent variables were that  $i$  equals 5 percent,  $n$  equals 13 yr (since construction must be concluded in 1985 for the comparison to be valid), and  $m$  equals 20 yr.

Since these assumptions were most certainly to be challenged, a thorough sensitivity analysis was performed for each assumption and procedures were developed for recomputing  $x$  using the challenger's own assumptions.

Due to the extreme difficulty of predicting construction costs in the future, three separate costs were developed for each gear type. An assumption was made that a probable coefficient of variation of 20 percent existed in both unit price and area to be paved calculations. Based on this assumption, the most probable equivalent annual pavement upgrading cost (MPC) was computed, a lowest probable cost (LPC) of pavement upgrading was computed assuming a 20 percent low-side calculation in both  $P$  and  $A$ , and a highest probable cost (HPC) was computed assuming a 20 percent high-side calculation in both  $p$  and  $a$ . In all three cases the original assumptions for  $i$ ,  $n$ , and  $m$  were not changed. The three equivalent annual pavement upgrading costs for each projected 1985 major hub airport for the Category I and II aircraft are shown in the following tabulation:

	LPC	MPC	HPC
Category I airplane:			
Median gear	\$12,013,910	\$33,328,803	\$ 65,324,506
Optimized gear	12,666,249	35,218,395	69,002,973
Category II airplane:			
Median gear	\$12,149,814	\$33,749,362	\$ 66,148,900
Optimized gear	20,902,029	58,097,736	113,842,221

It should be remembered that, while these analyses were performed for the pavement upgrading cost, only a single-point estimate of the aircraft penalty cost has been made. This factor should be considered in examining conflicting alternatives.

#### 15.6 SUMMARY OF FINDINGS AND CONCLUSIONS

Presented below are cost comparisons based on modifying and retaining the present design criterion. This presentation first considers only the Category I aircraft since the possibility exists that the Category II aircraft will not be operational in 1985.

##### 15.6.1 CATEGORY I AIRCRAFT

Based on the equivalent annual cost analysis using the MPC for pavement, the total equivalent annual costs are:

- a. Current gear: \$6,673,379.
- b. Median gear: \$35,258,683.
- c. Optimal gear: \$35,218,395.

It is obvious from this listing that the optimal alternative is not to modify the present policy if only the Category I aircraft are considered.

If the LPC is used for pavement, the decision remains unchanged as shown by these costs:

- a. Current gear: \$6,673,379.
- b. Median gear: \$13,943,790.
- c. Optimal gear: \$12,666,249.

##### 15.6.2 CATEGORY I AND II AIRCRAFT

The state of the art in pavement analysis is in its infancy concerning mixed traffic and pavement deterioration predictions. Therefore,

a basic assumption inherent in the following analysis is that a pavement structure upgraded for the Category II aircraft would be adequate for the additional Category I aircraft, concurrently. Based on the equivalent annual cost analysis using the MPC for pavement, the total annual costs are:

- a. Current gear: \$75,451,243.
- b. Median gear: \$70,840,062.
- c. Optimal gear: \$58,097,736.

Based on this total annual cost listing, the present design criterion should be changed to permit the optimization of the gear to the Category II aircraft.

However, in this instance, if the HPC is used for pavement, a conflicting alternative arises as shown by these projected costs:

- a. Current gear: \$75,451,261.
- b. Median gear: \$103,239,690.
- c. Optimal gear: \$113,842,221.

There is considerable logic behind the assumption that the MPC will be exceeded in the pavement upgrading for the Category II aircraft. In all probability, the paved area will exceed that computed in this study. The unit price differential may or may not increase. Thus, it is extremely critical to the decision-maker that a proper determination be made as to whether or not the Category II aircraft will be operational in 1985, whether or not it will operate at all 26 projected major hub airports or perhaps only at 7 to 10 regional airports, and other operational assumptions.

#### 15.7 RECOMMENDATIONS

The following recommendations are based on the results of this study. They are based solely on the calculations and assumptions presented. Devices were developed during the study to permit the decision to change these assumptions and calculations, and the possibility exists that the recommendations could change based on further developments.

- a. If only the Category I aircraft will be in operation at each of the 26 projected major hub airports in 1985, the current criterion should not be changed.

- b. If the Category I and II aircraft (implied also is the Category II aircraft alone) will be in operation at each of the 26 projected major hub airports in 1985, the current criterion should be changed to permit the gear to be optimized to the aircraft.
- c. The possibility of operating the Category II aircraft at from 7 to 10 regional airports should be investigated.

## CHAPTER 16: NONDESTRUCTIVE TESTING

### 16.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-73-205, "Nondestructive Vibratory Testing of Airport Pavements," Volume I, "Experimental Test Results and Development of Evaluation Methodology and Procedures," September 1975, and Volume II, "Theoretical Study of the Dynamic Stiffness and Its Application to the Vibratory Nondestructive Method of Testing Pavements," April 1975.

### 16.2 BACKGROUND

Current methods for evaluating the load-carrying capacity of airport pavements require direct sampling techniques that are both costly and time-consuming. Often, direct sampling requires the closing of a pavement facility to traffic operations, which in turn necessitates the rerouting and/or rescheduling of aircraft. With the number of traffic operations increasing rapidly, the closing of a pavement facility, even briefly, can result in inconvenience to the traveler and higher costs to the air carrier. Also, the increasing gross weights and/or increasing operations of aircraft make the need for accurate and frequent evaluations of pavement systems extremely important to the airport owner since many facilities will need strengthening or rehabilitation to meet this increased demand. Given these considerations, the need for a procedure that permits rapid evaluation with a minimum of disturbance to normal traffic operations is evident. The use of nondestructive testing techniques to determine the pertinent characteristics of pavement systems offers the best promise of serving this need.

### 16.3 PURPOSES

The purposes of this study were to:

- a. Select and prepare specifications for equipment for nondestructive testing of airport pavement systems.
- b. Develop a methodology for evaluating the load-carrying capacity of airport pavement systems using the selected equipment.

- c. Develop an evaluation procedure based on this methodology.
- d. Develop a mathematical model which describes pavement response to dynamic loading.

#### 16.4 SCOPE

Various types of nondestructive testing equipment were studied through comparison tests on a wide range of pavement systems. In the comparison tests, vibrator static weights, peak vibratory loads, methods of application of the vibratory load to the pavement (including load plate types and sizes), frequencies of loading, and mobility and ease of operation were considered.

Tests for all phases of the study were performed at nine airports and on WES test sections. Nondestructive data collected at these locations included DSM\* values, deflections for frequency sweeps from 5 to 100 Hz, deflection basin measurements, and wave propagation data. Direct sampling test data collected included the thicknesses of all layers of the material comprising the pavement sections, foundation strength values (CBR's or modulus of subgrade reaction k values), concrete flexural strengths, and material classifications. Environmental factors considered included temperature and frost effects. Rigid and flexible pavements were studied, and chemical and bituminous stabilization of layers of materials was considered.

#### 16.5 SUMMARY OF WORK ACCOMPLISHED

##### 16.5.1 SELECTION OF EQUIPMENT

Five vibratory testing devices were evaluated: the WES 16-kip vibrator (Figure 16-2); the Civil Engineering Research Facility nondestructive pavement test van; the WES 9-kip vibrator; the Dynaflect; and the Model 505 Road Rater. The results of the comparison tests were plotted in the form of DSM's obtained with the WES 16-kip vibrator versus DSM's obtained with each of the other vibrators, and the standard error of estimate was computed for each plot.

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\* The DSM (dynamic stiffness modulus) is the inverse of the slope of a load versus deflection curve. An example curve and DSM calculation are shown in Figure 16-1.

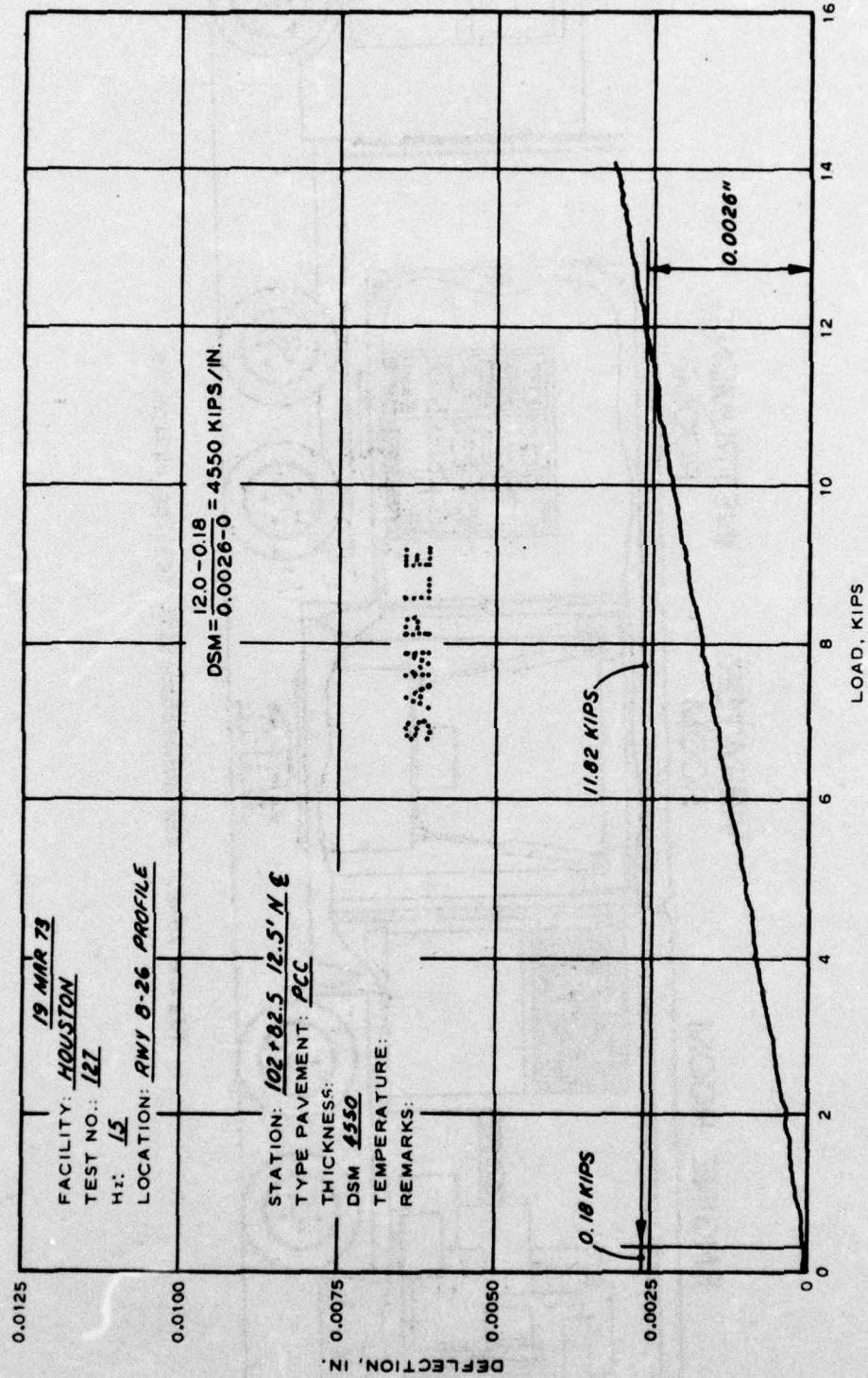


Figure 16-1. Deflection versus load (sample plot)

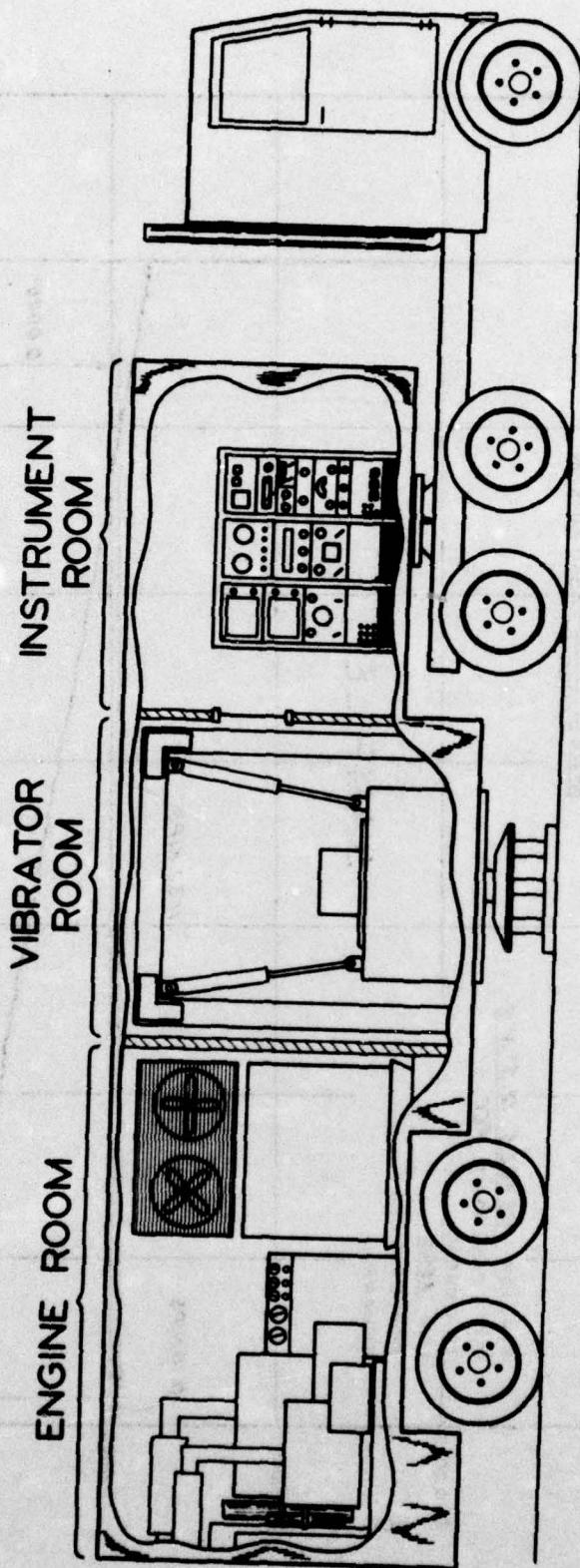


Figure 16-2. WES nondestructive testing equipment

The effects of vibrator static weight on load-deflection measurements were studied using a WES variable static weight 50-kip vibrator. Data taken on four different pavement items showed that the DSM increased significantly with increases in static weight applied to the pavement.

The effects of vibrator dynamic load on load-deflection measurements were studied with the WES 16-kip vibrator. It was found that the load versus deflection plots obtained with this equipment generally tended to curve at the lower force levels and become linear at higher force levels (10 to 15 kips), especially on weak flexible pavements.

Tests with the 16-kip vibrator also showed that the deflection responses of rigid and flexible pavements varied appreciably with changes in frequency. Earlier studies by WES indicated that 15 Hz was the optimum frequency for deflection tests because stress and deflection measurements with depth were greater for 15 Hz than for other frequencies within the capability of the vibrator.

Tests with the WES 16-kip vibrator using 12-, 18-, and 30-in.-diam load plates on flexible pavement test sections showed a pronounced effect on DSM values of changes in load plate diameter.

Accuracy and reproducibility tests with the WES 16-kip vibrator indicated that it is a reliable measuring device.

A recommended equipment specification for a variable-load nondestructive testing device was proposed based on the vibrator comparison tests.

#### 16.5.2 DEVELOPMENT OF EVALUATION METHODOLOGY

Tests for development of the evaluation methodology were conducted at the following facilities:

- a. National Aviation Facilities Experimental Center, Atlantic City, N. J.
- b. Houston Intercontinental Airport, Houston, Tex.
- c. Washington-Baltimore International Airport, Baltimore, Md.

- d. Greater Wilmington Airport, Wilmington, Del.
- e. Philadelphia International Airport, Philadelphia, Pa.
- f. Jackson Municipal Airport, Jackson, Miss.
- g. Nashville Metropolitan Airport, Nashville, Tenn.
- h. Weir-Cook Municipal Airport, Indianapolis, Ind.
- i. WES.

Pavements were characterized through direct sampling techniques and conventional testing methods. Prior to direct sampling, series of nondestructive tests were performed at each test site.

The effects of temperature on DSM measurements for flexible pavements were observed on a temperature test section constructed at WES. The observations resulted in the development of temperature adjustment factor curves which allow adjustment of DSM values to a constant mean temperature of 70° F.

The effects of freeze-thaw cycles on DSM measurements were observed at Truax Field in Madison, Wis. Core holes were drilled at test sites to determine pavement thicknesses and pavement and subgrade materials. The observations resulted in a graph of DSM versus time which showed an increase in deflection after the thaw began. The development of a correction factor to reduce loads determined by evaluation when pavements are not influenced by thaw was hampered by the complicated testing conditions and poor drainage properties of the subgrade materials.

Tests on rigid pavements showed that the DSM values could vary significantly from slab edges to centers. It was therefore decided that DSM tests should be performed at slab centers to obtain consistent results. Variations in DSM values on trafficked and untrafficked areas on flexible pavements indicated that DSM tests should be carefully located to accurately reflect the condition of the area to be evaluated.

The basic elements of the nondestructive evaluation methodology are described in the following subparagraphs:

- a. Flexible pavements. The nondestructive evaluation procedure for flexible pavement uses a measurement of the rigidity of the total pavement system (the DSM) and does not consider

the individual parameters that affect pavement response. The methodology revolves around establishing correlations between the DSM and an allowable single-wheel load. These correlations were developed for a single wheel having a tire contact area of 254 sq in. and for 1200 annual aircraft departures based on a 20-yr life. After the DSM versus allowable single-wheel load relation was developed, the methodology was based upon existing interrelationships between pavement thickness, load, load repetitions, soil strength, and landing gear characteristics.

- b. Rigid pavements. As with flexible pavements, the nondestructive evaluation procedure for rigid pavements uses a measurement of the overall rigidity of the total pavement system (the DSM). The DSM of a rigid pavement is a function of the pavement thickness and the concrete and foundation load-deformation characteristics. As for flexible pavements, the methodology is based on a correlation of DSM with an allowable single-wheel load. The relationship between the single-wheel load and gears of different geometry is based on the equivalency of maximum bending stress in the concrete slab.

#### 16.5.3 NONDESTRUCTIVE EVALUATION PROCEDURE

The complete evaluation procedures for rigid and flexible airport pavements are presented in Airport Pavement Bulletin No. FAA-74-1, "Nondestructive Testing Evaluation of Airport Pavements," dated September 1974.

Determinations of allowable multiple-wheel aircraft loads using DSM values require that the pavement thickness  $t$  be known for flexible pavements and that a foundation strength factor  $F_F$  and the pavement thickness  $h$  be known for rigid pavements. The required parameters can be determined from information in construction drawings or from tests in small core holes. Only one core hole is necessary for determining the parameter for a feature which has uniform properties. Pavements that contain chemically or bituminous stabilized layers can be evaluated by use of equivalency factors which convert the thickness of stabilized pavements to thickness of conventional pavement sections.

Because of the ease of making measurements, no less than 30 DSM tests should be made within each paved area for major features. A representative DSM value for each area is the arithmetic mean of the DSM's measured for the area minus one standard deviation.

A correction for the effects of freeze-thaw cycles on DSM data has not been developed, so DSM measurements should not be made when pavements are under the influence of frost. A correction for temperature effects on flexible pavements, however, has been developed. Mean pavement temperatures for use in calculating the correction can be determined by installing thermometers in a vertical line at 1 in. below the surface, at the center, and at 1 in. above the bottom of the surfacing layer and taking the mean of the three readings. Mean pavement temperatures can also be estimated by a method outlined in Asphalt Institute Manual Series No. 17, dated November 1969.

Data used in developing the correlations of DSM and gross aircraft weight were collected from areas of pavements which were free of surface deficiencies. Therefore, data collected for evaluation purposes should be from areas in good condition. For a feature where deficiencies exist, the allowable load should be reduced by a judgment factor.

#### 16.5.4 MATHEMATICAL MODEL

A mathematical model of the response of pavements subjected to dynamic loading was developed. This model is based on a theory of the nonlinear oscillator. In this model, the dynamic stiffness is described by the following characteristics: dynamic load, static load, frequency, load plate radius, and elastic moduli of each pavement layer. The elastic restoring force of the pavement is described by linear, cubic, and fifth-order terms in the displacement of the surface, so that three basic parameters are required by the model. These three parameters are determined from the dynamic load-deflection characteristics. The model uses a finite depth of influence which depends on the static load and the contact area of the vibrator load plate with the pavement surface. The nonlinear behavior of the dynamic load-deflection curves is due to the finite depth of influence. With increasing load plate sizes, the depth of influence increases and passes through the successive pavement layers.

## 16.6 SUMMARY OF FINDINGS AND CONCLUSIONS

A nondestructive evaluation procedure for flexible and rigid airport pavements and equipment specifications for a nondestructive testing device were developed. It was concluded that additional studies of vibrators, deflection measurements on composite pavements, theoretical relationships between vibrator data and allowable loads, relationships between deflection data and rigid pavement slab properties, relationships between deflection data and temperature, wave velocity techniques to determine elastic properties, relationships between deflection data and pavement performance, and the effect of pavement overlays on vibrator results were warranted to improve the confidence level of the evaluation.

The mathematical model describing the nonlinear response of pavements gives the capability of predicting the dynamic stiffness of a pavement for the loading conditions on the pavement directly under an aircraft wheel. The model also gives a simple analytical correlation between the different values of dynamic stiffness measured by different vibrators at the same pavement location. In addition, the model may yield a method for predicting the thickness and elastic moduli of each pavement layer in terms of the measured values of the dynamic stiffness for a series of load plate sizes.

## CHAPTER 17: SOIL CLASSIFICATION SYSTEMS

### 17.1 APPLICABLE PUBLICATIONS

This chapter summarizes Report No. FAA-RD-73-169, "Review of Soil Classification Systems Applicable to Airport Pavement Design," May 1974.

### 17.2 BACKGROUND

Soil scientists and engineers have established several classification systems for soils and rock materials. The intended use to be made of the resulting classification usually determines which system is used. Various groups of soil scientists and engineers have established classification methods which are tailored to provide information consistent with the requirements of their particular field of interest.

Pavement engineers are interested in the in situ or remolded strength, drainability, swell potential, pumping susceptibility, frost susceptibility, and other factors that affect the design of a pavement structure. Thus, they are interested in a soil classification that conveys this type of information and which will permit individual engineers to communicate in a common language concerning soil properties. It should also enable relatively uniform interpretations of the resulting classification. Nevertheless, within pavement circles, more than one soil classification system is in use, making communication among pavement engineers difficult.

Soil classification is such a useful tool that classification has become routine. It permits an estimate of the expected behavior of the soil at a given location--a factor of utmost importance in pavement design and construction.

### 17.3 PURPOSE

The purpose of this study was to determine whether the FAA should retain its present soil classification system or change to another system.

### 17.4 SCOPE

This study consisted of:

- a. A critical review of the American Association of State Highway and Transportation Officials (AASHTO), FAA, and Unified Soil Classification (USC) systems.
- b. Comparisons of the merits of the FAA system and the other two systems.
- c. The possible impact on the airport pavement industry of any changes in the FAA system was analyzed.

#### 17.5 SUMMARY OF WORK ACCOMPLISHED

The first action taken in the study was to send a questionnaire to individuals involved in the paving industry. The replies to the questionnaire were set aside during the analysis phase of this project, and the technical analyses and comparisons were performed without reference to the questionnaire replies.

Secondly, technical analyses and comparisons were performed that involved (a) definition and evaluation of each of the three classification systems and (b) statistical comparisons of the systems on the basis of their ability to predict the behavior of both fine- and coarse-grained soils. The accuracy with which the various soil properties are defined by the different systems was also compared statistically.

#### 17.6 SUMMARY OF FINDINGS

As a basis for the comparisons made in this study, the major requirements of a soil classification system were defined as: (a) soil strength, (b) drainability, (c) swell potential, (d) susceptibility to pumping, (e) frost susceptibility, and (f) compactibility. For adequate classification, the system used should also fulfill the following:

- a. The system should account for the macroproperties (areal) of a soil as well as the microproperties.
- b. It should permit prediction of the probable behavior of the soil under a variety of climatic, moisture, and loading conditions.
- c. The classification system should be simple enough to be used routinely but at the same time be rational and soundly based.
- d. The system should permit an estimation of a variety of soil properties but should not require extensive and complicated tests for the classification.

- e. The system should not define soil properties so broadly that a wide variety of values are permitted within a given classification group.
- f. The classification system should be capable of being well understood by the engineering profession.

It was also determined that the soil properties or characteristics that are most significant in predicting the probable behavior of a given soil are the plasticity index (PI), liquid limit (LL), grain size, and grain-size distribution. Also of importance, especially for prediction of strength, are moisture-density relationship, grain texture, grain shape, and other related factors.

Drainability is a direct function of the grain-size distribution (particularly the amount of fines) and the plasticity of the fines. Prediction of swell potential is heavily dependent on the established values of the PI.

Soil plasticity is the major indicator, insofar as predicting susceptibility to pumping is concerned.

The percentage of fines smaller than 0.02 mm (a value obtained from the grain-size distribution determination) and the PI are the major indicators of frost susceptibility.

Soil compactibility is dependent on several factors, but again the PI is the most significant.

#### 17.7 CONCLUSIONS

A comparison of the three classification systems (AASHTO, FAA, and USC) and analysis of the questionnaires (almost 100 percent were returned) led to the following conclusions:

- a. The AASHTO and USC systems utilize both liquid and plastic limits and thus are better descriptors of soil behavior than the FAA system which uses only the liquid limit.
- b. The AASHTO and USC systems more accurately describe the probable strength of the soil than the FAA system, with the USC system being the best predictor of strength.
- c. The USC system is the better predictor of the frost susceptibility of the soil.
- d. The USC system permits a closer estimate of the susceptibility to pumping of soils than the other two systems; however, all three are deficient in this regard.

- e. The USC system approaches an adequate prediction of permeability of soil whereas the other two systems are inadequate in this respect.
- f. The AASHTO and USC systems predict the variances that might be expected in a given soil deposit with a greater precision than the FAA system with the USC system being the best of the three systems in this regard.
- g. Consultants, representatives of the materials industry, and airline personnel preferred the USC system over the other two systems. FAA regional offices gave almost equal preference to the FAA and USC systems.
- h. A majority of people questioned indicated that the USC system was more satisfactory in its present form than either the AASHTO or FAA systems. A majority felt that the AASHTO and FAA systems needed major revisions while only a few believed that the USC system needed such action.
- i. Although unsolicited, by far the majority of respondents felt that the pavement design and evaluation should be based upon soil testing rather than upon soil classification.

#### 17.8 RECOMMENDATIONS

On the basis of the analyses of data, particularly in light of the demonstrated levels of sensitivity of the three classification methods, and considering the variability that exists in the field, it was recommended that the FAA adopt the USC system for classification of soils.

Pavement design based on soil classification has the disadvantages of not accounting for soil strength, compactibility, moisture content, and several other factors. This fact, coupled with the reactions received in the questionnaires, leads to the recommendation that the FAA develop new design procedures based on strength criteria. Classification would thus become a secondary, rather than a primary, factor in the design procedure.

The implementation required for adopting the USC system should present no problems to the FAA. The USC system has been in widespread use by the engineering profession since about the mid-1940's. Engineering colleges with accredited courses in soil mechanics teach the method routinely. Soils and paving engineers have been using the method

for more than 20 years, and there should be no difficulty in adapting to the new system. In fact, the results of the questionnaire suggest that many engineers are looking forward to the change, should it be made.